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The top and bottom stories sway subassemblages analysis

Shih Hsun Yuan
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A Thesis
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CERTIFICATE OF APPROVAL

This thesis is accepted and approved in partial fulfilment of the requirements for the degree of Master of Science in Civil Engineering.

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Abstract

The sway subassemblage method of analysis separates the structure into basic units called subassemblages. By analyzing the elastic-plastic behavior of subassemblages, this method provides an approximate way to develop the lateral-load versus sway-deflection curve for a story.

The method has already been developed for stories in the mid height part of an unbraced frame. An assumption is made that the inflection point is at the mid-height of the column for the purpose of analysis, but top and bottom stories have different boundary conditions from interior stories. Also the bottom story can be divided into two situations as hinged base and fixed base according to its foundation. The main objective of this thesis is to develop the method of handling top and bottom stories of buildings on the basis of the subassemblage method.

A computer program written in FORTRAN "SMOA" is based on the sway subassemblage method. The program reads the input file data which are from the preliminary design. After operating on each subassemblage by computer, the output file gives the load-deflection curve of the story and shows the sequence of forming hinges in each member until the mechanism is formed for each subassemblage in the story. The results can be used as a check for a preliminary design and make a revision to the design. A lot of improvement has been made to SMOA". Now the program is able to be executed on either a main frame computer or a microcomputer. The program can determine the increment of the angle for the formation of the next hinge in the subassemblage until a mechnism is formed, and this will save a substanial amount of computer time.

Chapter 1
Introduction

1.1 Background

When an unbraced frame is subjected to combined gravity and wind loads, in addition to the stress directly cause by these loads, an additional moment will occur as the p-Δ moment. The horizontal loads push the building frame into a sway which make a horizontal distance between the top and bottom for each story. The gravity loads amplified by distance Δ will produce secondary moments. These moments have been found to cause significant reductions in frame strength and must be accounted for in design procedures (Hansell, 1966).

The method is based on the concept of sway subassemblages and restrained columns with sway and is used to develop the lateral load versus sway deformations relationship for a story. The members are selected from preliminary design (Driscoll et al, 1965) and the vertical load on each column has been calculated previously. The whole structure is analyzed by stories. Each story is divided into subassemblages, which consists of a column and one or two adjacent girders. Each subassemblage is analyzed for its lateral load-sway deflection behavior, then all the load deflection curves of the subassemblages in the same story are combined to determine the load-deflection curve for the story. The member sizes can be determined on the basis of maximum strength or deflection.

1.2 Scope of Development

The subassemblage method which has already been developed deals with the stories in the mid-height part of buildings. An assumption has been made that the inflection point above or below a subassemblage is at mid-height position of the columns. For a real situation, the inflection points of a story in the middle part of a high level building, will be about at the mid-height place of columns. There would be some distance difference variation with the stiffness of beams and columns in the building, but the errors are small enough to be neglected (Parikh, 1966). When dealing with the top and bottom stories, the assumption of inflection points at the mid-height of columns is not necessarily valid, since there will be no stiffness above the subassemblages of top stories, and for the bottom stories, the foundation can be viewed as a fixed base or a hinged base according to the type of foundation. The subassemblages will behave in a different way from those in the middle part of a building.

A method will be introduced to handle the situations for the subassemblages of top and bottom stories. A special case of the building with only one story will also be discussed. One-story buildings have different properties from either top, bottom or middle stories, and they are even more popular than tall buildings in the United States.

A computer program written in FORTRAN is discussed in the report. Some improvement has been introduced to make the program execute more efficiently and it can be used for more situations and choices. An example in the report will show how the program works.

Chapter 2
The Subassemblage Method Of Analysis

2.1 General Definition

2.1.1 The Role of Subassemblages

There are two types of subassemblages, sway and nonsway subassemblages (Levi, 1964). For an unbraced frame with an additional moment to the members caused by the p-Δ effect, an effective way to analyze the structure is using the sway subassemblage method. The sway subassemblage method separates the whole frame into individual units and analyzes the behavior of the members in each unit.

Consider an unbraced multi-story frame building as shown in Fig.2-1. The stories in the middle or lower part of the building will be subjected to combined loads composed of gravity load and wind load. In the preliminary design, it is assumed that the columns bend in double curvature and their inflection points are at the mid-height. With this assumption, each story can be divided from the frame at the inflection points above and below the level as shown in Fig.2-2. Then each level can be further separated into sway subassemblages. Each subassemblage has a restrained column as the main part of the subassemblage. It is connected to one or two girders depending on whether it is a side column or not. The sway subassemblages are analyzed by statics and the relationship of load and deflection for each subassemblage. Then the results of all subassemblages are combined to determine the behavior of the story.

2.1.2 Sign Conventions

The sign convention for analyzing subassemblages will be based on the character of the sway subassemblage under combined loads. When a frame is only subjected to gravity loads, the moment distribution of the girders in the frame will be somewhat like Fig.2-3a. If the frame is only subjected to wind load coming from the left direction, the moment distribution would be like Fig.2-3b. The moment at the left end of the girder is initially counterclockwise. As the wind load from the left direction is increased, the moment will become smaller, then become clockwise and the wind load will also cause the moments from gravity load to increase on the right end of the girder. If the wind load is large enough, the final moment distribution for a girder in a frame subjected to combined loads will be like Fig.2-3c. Since for the case of an unbraced frame under combined load, the moment distribution for girders usually will act as in Fig.2-3c, these directions of moments on the ends of girders will be used as the basis of sign conventions. The sign conventions for the columns are also the same. The rules of the sign conventions are

1. Moments and rotations at the ends of members are positive when clockwise.
2. Internal moments of a joint are positive when countclockwise.

2.2 Method of Analysis

2.2.1 Behavior of Beams and Columns

2.2.1.1 Restrained Column in a Sway Subassemblage

Consider the story n in Fig.2-1 which is isolated from a frame by the inflection points. The story contains the girders at level n and the column portions between the inflection points right above and below the story n. The free body diagram of the story n is shown in Fig.2-2.

A story assemblage like this can be divided into four subassemblages as in Fig.2-4. A general or interior subassemblage will be as in Fig.2-5. Depending on whether it is at right or left end of the frame, the column on the edges of the story has only one girder at its left or right side.

To analyze the subassemblage structure, the subassemblage can be further modeled as shown in Fig.2-6a. A column is restrained by a rotational spring at the top, and the bottom is located on a hinged base. This model can be used to describe the behavior of the subassemblage. The spring represents the girders which are connected the to column and has the properties of the same rotation stiffness as the girders. If a unit rotation is applied at the top of the column, the resisting moment of the girders will be represented by the spring force F which is the stiffness K of the spring multiplied by the rotation angle. The hinged base stands for the inflection point which is subjected to vertical and horizontal forces but no moment. The vertical force is the axial force and the horizontal force is the shear force of the column. As mentioned previously, an assumption has been made that the inflection point is at the mid-height of the column, so $\frac{h}{2}$ would be the height of the model.

Referring to Fig.2-6b, and taking moments about the hinged base, an equilibrium equation can be obtained as

$$M_n = -(\lambda \sum H_n)\frac{h}{2} + P_n \frac{\Delta_n}{2} \tag{2.1}$$

This eqation considers the effect of P-Δ effect. Also in Fig.2-6b three rotations, $\frac{\Delta}{h}$, θ, and γ, are measured clockwise as they are shown in Fig.2-6b. Notice that arrow direction of γ is negative in the figure.

According to the angle relationship in Fig.2-6b, a compatibility equation can be written as

$$\frac{\Delta_n}{h} = \theta - \gamma \tag{2.2}$$

Again from this figure, the equilibrium of moments at the upper joint leads to the equation

$$M_r = M_{n-1} - M_n \tag{2.3}$$

By a conservative estimate (Daniels, 1966a), M_{n-1} is assumed to equal to M_n, equation (2.3) is changed to

$$M_r = -2M_n \tag{2.4}$$

The load-deflection relationship of the restrained column can be determined by solving eqations (2.1), (2.2), and (2.4) with the moment-roration relationship of the column (Parikh, 1965).

2.2.1.2 Restraining Girder in the Sway Subassemblage

Consider again the subassemblage in Fig.2-5. The column in the subassemblage is constrained by the adjacent girders which can be modeled as a spring restraining the column as shown in Fig.2-6a. A girder which is connected to two columns is used not only in one subassemblage but in the subassemblages on both sides of the column. The restraining coefficients will be affected not only by one subassemblage but also should be affected by the behavior of the whole structure. For the analysis of subassemblages, it is not necessary to consider the influence from the whole structure, because the other parts of the structure will have relatively smaller influence on the restraining characteristics of the subassemblage than the structure on its sides. For design purposes, a consideration of the restraint at the far end of a girder by the restraining effect of the column and girder which are directly fastened to that end is enough. For the subassemblage B'-A-B in Fig.2-7, to determine the restraining coefficients of girder A-B, the right part of the structure shown in Fig.2-8 will give equations of the restraining coefficients.

$$K_{AB} = 6\left[\frac{3 + 0.5\beta + \eta + \alpha' \dfrac{K_{AB}'}{12}}{3 - 0.5\alpha + \beta + 1.5\eta}\right] \tag{2.5}$$

where,

$$\alpha = \frac{h_A\ I_{AB}}{L_{AB}\ I_A}\left[\frac{1 + 2\dfrac{d_A}{L_{AB}}}{1 + \dfrac{d_{AB}}{h_A}}\right] \tag{2.6}$$

$$\alpha' = \frac{h_A\ I_{AB'}}{L_{AB'}\ I_A}\left[\frac{1 + 2\dfrac{d_A}{L_{AB'}}}{1 + \dfrac{d_{AB'}}{h_A}}\right] \tag{2.7}$$

$$\beta = \frac{h_B\ I_{AB}}{L_{AB}\ I_B}\left[\frac{1 + 2\dfrac{d_B}{L_{AB}}}{1 + \dfrac{d_{AB}}{h_B}}\right] \tag{2.8}$$

$$\eta = \frac{h_B\ I_{BC}}{L_{BC}\ I_B}\left[\frac{1 + 2\dfrac{d_B}{L_{BC}}}{1 + \dfrac{d_{BC}}{h_B}}\right] \tag{2.9}$$

By the slope-deflection equation, some other restraining coefficients can be obtained.

$$K_{BA} = 4 \left[\frac{3 - K_{AB}}{4 - K_{AB}} \right] \tag{2.10}$$

$$K_{AB'} = 4 \left[\frac{3 - K_{B'A}}{4 - K_{B'A}} \right] \tag{2.11}$$

where $K_{B'A}$ uses the same form as equation (2.5). The details of derivation for the restraining coefficients can be seen in Daniel's report (Daniels, 1967).

With increasing sway, the moment diagram of the girders will tend to have a shape like Fig.2-3c and the column will deflect more with the sway. When the moment exceeds the plastic moment capacity of the member, plastic hinges will be formed at critical places. According to Fig.2-9, we can see some locations are more likely than others to form a hinge. These places are marked as 1, 2, 3, 4 and C which are at the ends of the girders and at the column top. Notice that sometimes the largest moment near the left end of the girder is not right at the end, it could occur at a point of zero shear between the mid-point and the left end. These points are labelled X and Y in the figure. These are also the symbols used in the program SMOA. The hinges at 2 and X or 4 and Y won't occur simultaneously.

When a hinge is formed at any end of a girder, the restraining coefficients of the girder will change. The values will change to either 3 or zero depending on the location of the plastic hinge. (Daniels, 1966a)

Referring to Fig.2-9 again, when lateral load is applied to the frame, the initial sway produces a rotation θ_A at the top of the column. By the slope-deflection equations, relative values of θ_B and $\theta_{B'}$ are

$$\theta_B = \frac{K_{AB} - 4}{2} \theta_A \tag{2.12}$$

$$\theta_{B'} = \frac{K_{AB'} - 4}{2} \theta_A \tag{2.13}$$

Then the moment changes at the ends of the girders will be

$$\delta M_{AB} = K_{AB} \frac{EI_{AB}}{L_{AB}} \theta_A \tag{2.14}$$

$$\delta M_{AB'} = K_{AB'} \frac{EI_{AB'}}{L_{AB'}} \theta_A \tag{2.15}$$

$$\delta M_{BA} = K_{BA} \frac{EI_{AB}}{L_{AB}} \left[\frac{K_{AB} - 4}{2}\right] \theta_A \tag{2.16}$$

$$\delta M_{B'A} = K_{B'A} \frac{EI_{AB'}}{L_{AB'}} \left[\frac{K_{AB'} - 4}{2}\right] \theta_A \tag{2.17}$$

To the initial moments of the girder add the difference of moment due to the change of θ_A. Then the moment distribution in girders can be determined. Notice that the moment at any point can not exceed the plastic moment capacity of the girder section.

2.2.2 Mechanism of Subassemblages

When a subassemblage forms a mechanism, it means the upper end of the column can rotate arbitrarily without applying additional forces. In other words, it loses its stiffness.

There are two conditions by which a subassemblage loses its stiffness. In the upper end of the column forms a hinge; in the other both of the ends of the girders near the column form hinges. Then the subassemblage loses its ability to resist additional loads. The first plastic hinge usually forms at the leeward end. So an interior subassemblage usually needs three hinges on the girders to form a mechanism. The windward subassemblage needs two hinges, and the leeward subassemblage needs only one hinge to form a mechanism. For example, the leeward subassemblage has only one girder on the left side of the column. As mentioned before, the leeward end of a girder forms a hinge more readily than the other end. Once a hinge has formed at the leeward end of the leeward girder, because there is no girder on the right side of the column, the subassemblage becomes a mechanism immediatelly. For the case of a windward subassemblage, it has one girder at the right side. Usually a hinge will be formed at the leeward end before the formation of the hinge at the windward end, so two hinges are needed to form a mechanism. The same concept can also be applied to the interior subassemblages.

2.2.3 Load - Deflection Relationship

2.2.3.1 Load-Deflection Curve of a Subassemblage

The load-deflection curve is determined by the relationships of restraining girders and restrained columns as mentioned in previous sections (Daniels, 1966a). Consider the initial restraining moment M_r of the subassemblage in Fig.2-9.

$$M_r = \left[K_{AB'} \frac{EI_{AB'}}{L_{AB'}} + K_{AB} \frac{EI_{AB}}{L_{AB}} \right] \theta_A \tag{2.18}$$

The stiffness in equation(2.18) can be written with the term M_{pcA}(reduced plastic moment capacity) of the column as the initial restraining function M_{r1}.

$$M_{r1} = K_1 \ \theta_A \ M_{pcA} \tag{2.19}$$

where,

$$K_1 = K_{AB'} \frac{EI_{AB'}}{L_{AB'} M_{pcA}} + K_{AB} \frac{EI_{AB}}{L_{AB} M_{pcA}} \tag{2.20}$$

θ_A is increased up to a certain value when the first hinge is formed in the subassemblage. M_{r1} is its maximum value at that time and would be called M_{r1}'. Then until the second hinge is formed, the restraining function is

$$M_{r2} = K_2 \ \theta_A \ M_{pcA} \tag{2.21}$$

K_2 is in the same form as K_1, but the value of $K_{AB'}$ or K_{AB} may be changed because of the first hinge. Similarly, when the second hinge is formed, the restraining function becomes its maximum value $M_{r2'}$. The same procedure repeats until the subassemblage forms its mechanism. The curve can be constructed with the aid of design charts in references (Daniels, 1966b) (Parikh, 1965).

Fig.2-10 shows three typical load-deflection curves for windward, leeward and interior subassemblages. As mentioned in the previous section, the curve in Fig.2-10a. has two changes of slope which represents two hinges formed before a mechanism forms in the windward subassemblage. Leeward and interior subassemblages need one and three hinges to form a mechanism respectively.

2.2.3.2 Load-Deflection Curve of a Story

The load-deflection curve of a story is constructed from the load-deflection curves of the subassemblages in the story. The values of lateral load for all subassemblages are added at each deflection as shown in Fig.2-10. Fig.2-11 shows a typical load-deflection curve of a story. The curve starts at the origin point and increases at a slope corresponding to elastic behavior until a hinge is formed at some place in the story. Then the curve bends to a smaller slope because the story has less stiffness to resist the lateral load. As the lateral load keeps increasing, the curve decreases in slope more and more as more hinges are formed in the story. At the point when all the subassemblages in the story have became a mechanism, the story has no stiffness to resist more lateral loads. Then the story becomes a mechanism and the curve reaches its maximum point which is the ultimate shear force the story can resist.

Chapter 3
The Method Of Handling Top And Bottom Stories

3.1 General

As mentioned previously, the boundary conditions for top and bottom stories are different from those for interior stories. The assumption of the inflection point at the mid-height of the column is not good for the subassemblages of top and bottom stories. A more accurate position of the inflection point should be found to make the moment-deflection calculations of the subassemblages more accurate.

Because the wind loads are treated as concentrated loads on the joints for analysis purposes, the moment and shear diagrams of the column can be known if the moments on both ends of the column are known, and then the position of the inflection point can be determined. For a subassemblage in the whole structure, it's not easy and necessary to get the real values of moments of the two ends for each column. The moment distribution method is one of the ways to determine the ratio of the moments on the two ends which can be used to determine the location of the inflection points. Here a method called cantilever moment distribution method will be used which is more suitable and powerful than normal moment distribution method in this case.

3.1.1 The Cantilever Moment Distribution Method

The cantilever moment distribution method is modified from the ordinary moment distribution method to provide more rapid results for some special cases.

Consider a frame as shown in Fig. 3-1a, with a lateral force P applied on the left corner of the frame as shown in Fig. 3-1b. Because the columns are restrained by the girder, the shape of the frame will become anti-symmetrical as is shown in Fig.3-1b. Assume this is a symmetrical structure. For this case it means that the two columns have the same section properies, so the shape will become anti-symmetrical after the load P is applied.

Now consider the mid point of the girder. As an anti-symmetrical structure, it will remain at the same height as the frame without P acting on it and will have zero moment. This property is just like the function of a roller. The load P can be divided into two $\frac{P}{2}$ loads on both joints on the girder and columns which will make not only the geometry but also the load anti-symmetrical. Then cut the frame into two halves. Take the left portion and put a roller at the end of the girder as shown in Fig. 3-2a. For the horizontal shear in the column induced by a lateral displacement of the column top without end rotation.

By the concept of mechanics of materials, an equation

$$\frac{12EI_c\ \Delta}{L_c^{3}} = \frac{P}{2} \qquad (3.1)$$

where Δ and L_c are shown in the figure. E is the young's modulus. I_c is the moment of inertia of the column section.

The fixed end moments F_{ab} and F_{ba} of the column in equilibrium with the shear of Eq. 3.1 are

$$F_{ab} = F_{ba} = -6 \frac{EI_c \Delta}{L_c^2} \tag{3.2}$$

Eq. 3.1 may be solved for Δ and the result substituted into Eq 3.2 to obtain an alternate expression for the fixed end moments.

$$F_{ab} = F_{ba} = -P \frac{L_c}{4} \tag{3.3}$$

The main difference between traditional moment distribution and the cantilever moment distribution method is the treatment of the behavior after release of the displaced column top. Rotation of the column top to balance the fixed end moment is allowed to occur in combination with an additional lateral sway such that the change in moments is as shown in Fig. 3.2b. The stiffness of the cantilever against rotation at B is $\frac{EI_c}{L_c}$. In combination with the reduced stiffness of the propped cantilever beam BC. The stiffness ratio of the column and beam is

$$\frac{EI_c}{L_c} : \frac{3EI_b}{\frac{L_b}{2}} = \frac{EI_c}{L_c} : \frac{6EI_b}{L_b} \tag{3.4}$$

For the case in Fig.3-1. Assume the girder and columns have the same values of L, E and I. Then

$$K_{ab} : K_{bc} = \frac{EI}{L} : \frac{6EI}{L} = 1 : 6 \tag{3.5}$$

$$F_{ab} = F_{ba} = -\frac{PL}{4} \tag{3.6}$$

The moment distribution tabulation can be seen in Table.3-1. Since the structure has been considered as an anti-symmetrical structure, the moment distribution of the right portion would be anti-symmetrical to the left portion. The final moment distribution will also be anti-symmetrical as shown in Fig.

3-1c. The cantilever moment distribution method converges rapidly because one cycle is sufficient. No moment is reflected from the roller end of the beam, and only a single carryover of $-M_1$ is made from the top to the bottom of the column.

The rules of the cantilever moment distribution method are :

1. Find the fixed end moments on the ends of the columns
2. The modification coefficient of the stiffness rigidity is 6 for the girders and 1 for the columns.
3. The moment carry over factor for the column ends is -1.
4. Moment at ends of members are positive when clockwise.

The cantilever moment distribution method also has its restrictions.

1. The number of spans for the frame must be odd.
2. The frame has to be in a symmetrical shape; the stress and strain of the frame are anti-symmetrical.

Another example of a three-floor frame structure is shown in Fig.3-3a. Assume the frame is subjected to wind load from the left concentrated on the joints. The lateral load on the top joint is the half amount of the loads on the other joints because only half wind effect from the story would react on the top joint. The properties of the members are shown in the figure. Under the rules of the method,

$$K_{DC} : K_{DE} = \frac{EI}{3} : \frac{2EI}{6} \times 6 = 1 : 6 \tag{3.7}$$

$$K_{CD} : K_{CF} : K_{CB} = \frac{EI}{3} : \frac{4EI}{6} \times 6 : \frac{EI}{3} = 1 : 12 : 1 \tag{3.8}$$

Also

$$K_{BC} \; : \; K_{BG} \; : \; K_{BA} \; = \; 1 \; : \; 12 \; : \; 1 \tag{3.9}$$

The fixed end moments in each story result from the sum of the shear above.

$$F_{DC} = F_{CD} = -0.25 \times 12 \times 3 = -9 \tag{3.10}$$

$$F_{CB} = F_{BC} = -0.25 \times 36 \times 3 = -27 \tag{3.11}$$

$$F_{BA} = F_{AB} = -0.25 \times 60 \times 3 = -4.5 \tag{3.12}$$

The tabulation is shown in Tab.3-2. Considering the anti-symmetry, the final moment distribution diagram is plotted as Fig.3-3b.

3.2 Top Story

Fig.3-4 shows the upper part of a building and the deformed shape if the lateral load is applied to its left side. For each subassemblage in the top story, a general model is shown in Fig.3-5 including the stiffness of the column and girders. The value of $\frac{EI}{L}$ for one of the girders may be zero when the subassemblage includes an exterior column.

The main purpose of the model is to find the position of the inflection point in the column. When using the moment distribution method, the two stiffness of the beams can be combined to use $\frac{EI_b}{L_b}$ to represent $\frac{EI_{b1}}{L_{b1}}$ plus $\frac{EI_{b2}}{L_{b2}}$.

The subassemblage of Fig.3-5 then can be modeled as a half part of a top story in a single span building with $\frac{EI_b}{L_b}$ and $\frac{EI_c}{L_c}$ for the girder and columns respectively. The left part is taken for the analysis as the bold line in Fig.3-6.

Using the cantilever moment distribution method. According to Fig.3-6. The distribution factors and fixed end moments are

$$D_{BA} = \frac{\frac{EI_c}{h}}{\frac{EI_c}{h} + \frac{6EI_b}{L_b}} \tag{3.13}$$

$$D_{AB} = \frac{\frac{EI_c}{h}}{\frac{2EI_c}{h} + \frac{6EI_b}{L_b}} \tag{3.14}$$

$$D_{AD} = D_{AB} \tag{3.15}$$

$$F_{BA} = -\frac{Ph}{4} \tag{3.16}$$

$$F_{AB} = -\frac{Ph}{4} \tag{3.17}$$

$$F_{AD} = -\frac{3Ph}{4} \tag{3.18}$$

The tabulation of the moment distribution is in Tab.3-3. The subassemblage model of the top story column is in Fig.3-7. The position of inflection point from the top of the column is represented as αh, from the ratio of the top column moment to the sum of the two end moments.

where,

$$\alpha = \frac{1}{2} + 2\ D_{AB} - \frac{1}{2}\ D_{BA} \tag{3.19}$$

The cantilever moment distribution method converges rapidly (because the distribution factors of the columns are so much smaller than those of the pin-ended beams). The changes in column end moments are small enough that neglecting the influence of other parts of the structure below the top story will not cause large errors.

Using the three-story example in section 3.1.1 as a check, substitute the values of distribution factors in equation(3.19).

$$\alpha = \frac{1}{2} + 2\ \frac{1}{14} - \frac{1}{2} \cdot \frac{1}{7} = 0.5714 \tag{3.20}$$

the accurate value

$$\alpha = \frac{10.02}{10.02+7.26} = 0.5798 \tag{3.21}$$

The error is about only 1.4 percent.

Because there is no stiffness above the top story, the top girder has to take more moment; therefore, the inflection point will always below the mid point of the column in the top story.

Two assumptions are made for the analysis.

1. The girder of the second floor below the top has about the same stiffness as the top story girder.
2. The columns of the second floor have about the same stiffness as the top columns.

If the section properties of members between two adjacent floors have large differences, then the moment distribution would be subject to larger errors and lose its accuracy. In the real situation, the member sizes of adjacent floors usually wouldn't be too much different in a tall building.

After obtaining the value of α, the model in Fig.3-7 can be used to represent a subassemblage in the previous chapter as in Fig.2-6. Then the equations can be revised as follows

$$M = -\ (\ Q\ (\alpha h) + P\ (\beta\Delta)\ ) \tag{3.22}$$

$$M + M_r = 0 \tag{3.23}$$

$$\frac{\beta\Delta}{\alpha h} = \theta - \gamma \tag{3.24}$$

$$\theta = \frac{M_r}{M_{pc}} \gamma' \tag{3.25}$$

γ' is the rotation at $\frac{M}{M_{pc}} = 1$ which can be obtained in reference (Parikh, 1965). Since the deflected shape is somewhat close to a straight line, if a straight line is used to approximate the shape fram a to b, which makes $\alpha = \beta$

then,

$$M = -\alpha\ (\ Qh + P\Delta) \tag{3.26}$$

$$\frac{\Delta}{h} = \theta - \gamma \tag{3.27}$$

By the equations (3.23), (3.25), (3.26) and (3.27), the load-defleciton curve can be determined.

3.3 Bottom Story

The bottom story has two types of base. One is a hinged base, the other is a fixed base. The type of the base depends on the type of the foundation of the building. Usually a shallow type foundation such as a spread footing will be viewed as a hinged base and a deep foundation such as a pile foundation will be handled as a fixed base.

3.3.1 Hinged Base

Fig.3-8a shows the lower part of a building. If the wind load is applied to its left side, the deformed shape of the bottom story would be as in Fig.3-8b. Using the same concepts as in the previous sections, each subassemblage in the story can be modeled as in Fig.3-9.

For a simply supported beam in Fig. 3-10, the moment needed to rotate a unit angle at one end is $\frac{3EI}{L}$ which is inversely proportional to the length L. So if M_n is M, then M_{n-1} is 2M. Consider the angle relationships and the moment equilibrium of the subassemblage and at the base, write the equations as follows

$$M = -\left(Qh + P\Delta \right) \tag{3.28}$$

$$3M + M_{\gamma} = 0 \tag{3.29}$$

$$\frac{\Delta}{h} = \theta - \gamma \tag{3.30}$$

$$\theta = \frac{M_{\gamma}}{2M_{pc}} \gamma' \tag{3.31}$$

γ' can be determined in reference (Parikh, 1965). Then the relationships of the load-deflection curve can be determined by the four equations.

3.3.2 Fixed Base

The lower part of a building with fixed base and the deformation of the structure when wind load comes from the left direction are plotted in Fig.3-11.

A general model of the subassemblages in the bottom story is in Fig.3-12. An assumption has been made that the column above the subassemblage has the same stiffness properties as the fixed column.

As has been discussed in the previous section, the subassemblage in Fig.3-12 can be modeled as half of a single span frame with bolder lines as shown in Fig.3-13, where $\frac{EI_b}{L_b} = \frac{EI_{b1}}{L_{b1}} + \frac{EI_{b2}}{L_{b2}}$.

The moment distribution factors and fixed end moments are

$$D_{AB} = 0 \tag{3.32}$$

$$D_{BA} = \frac{\frac{EI_c}{h}}{\frac{2EI_c}{h} + \frac{6EI_b}{L_b}} \tag{3.33}$$

$$D_{BC} = D_{BA} \tag{3.34}$$

$$F_{BC} = F_{CB} = -\frac{1}{4}\left(n - \frac{3}{2}\right) P\ h \tag{3.35}$$

$$F_{BA} = F_{AB} = -\frac{1}{4}\left(n - \frac{1}{2}\right) P\ h \tag{3.36}$$

where n is the number of the story. The terms $n-\frac{1}{2}$ and $n-\frac{3}{2}$ occur because there is only $\frac{P}{2}$ lateral force applied to the top joint. The tabulation of the moment distribution is in Tab.3-4. As shown in the moment diagram in Fig.3-14, the distance from the top of the column to the inflection point is αh.

where,

$$\alpha = \frac{1}{2} - \frac{n-1}{n-\frac{1}{2}} D_{BA} \tag{3.37}$$

Take the three-story example in section 3.1.1as a check. When n=3, $D_{BA} = \frac{1}{14}$. Substitute these two values into equation(3.37), resulting in α=0.44286. The actual value of $\alpha = \frac{39.86}{50.14+39.68} = 0.44178$. The error is only 0.2 percent.

However, the greater the number of stories, the closer the term $\frac{n-1}{n-\frac{1}{2}}$ will approach 1. For a ten story building, the term is equal to 0.9524. For computer analysis purposes, 0.95 will be taken as a mean value. Then the equation becomes

$$\alpha = \frac{1}{2} - 0.95\ D_{BA} \tag{3.38}$$

The subassemblage model for this case is given in Fig.3-15. The same type of equations can be established as

$$M_n = -[Q(\alpha h) + P(\beta\Delta)] \tag{3.39}$$

$$2M_n + M_r = 0 \tag{3.40}$$

$$\gamma = \frac{M_r}{2M_{Pc}}\gamma' \tag{3.41}$$

$$\frac{\beta\Delta}{\alpha h} = \theta - \gamma \tag{3.42}$$

Now because of the fixed base, a straight line will not fit too well for the deflected shape of the column. A approximate formula is developed to achieve a better value of β. A curve normalizing the shape of the column in coordinates is in Fig.3-16. The equation of the curve is set as

$$y(x) = ax^3 + bx^2 + cx + d \tag{3.43}$$

Using the boundary conditions $y(0)=0$, $y'(0)=0$, $y''(\zeta)=0$, $y(1)=1$, then $d=0$, $c=0$, $a=\frac{1}{1-3\zeta}$, $b=\frac{-3\zeta}{1-3\zeta}$. The displacement η at the inflection point ζ is

$$\eta = \frac{2\zeta^3}{3\zeta-1} = 1 - \beta \tag{3.44}$$

where $\zeta = 1 - \alpha$. Combining these equations leads to

$$\beta = \frac{2\zeta^3-3\zeta+1}{1-3\zeta} \tag{3.45}$$

Use equation (3.39), (3.40), (3.41), (3.42) and (3.45) to get the load-deflection curve.

3.4 Special Case

Many buildings have only one floor on the ground. It is both a top and a bottom floor. This case has different boundary conditions from others, so it will be discussed separately.

3.4.1 Hinged Base

Fig.3-17. shows a one-story frame with hinged base and the deformed shape of the structure when the wind load comes from the left. The subassemblage of this case is somewhat like the subassemblage of the bottom story with a hinged base. The only difference here is that there is no stiffness above the girder in the frame.

The same Fig.3-9 discussed in section 3.3.1 can also be used here. The difference is that only one floor provides the restraining moments , and only columns below the girders need restraint. The restraint equations can be modified using M in place of $3M$ giving the following equations

$$M = -(Qh + P\Delta) \tag{3.46}$$

$$M + M_r = 0 \tag{3.47}$$

$$\frac{\Delta}{h} = \theta - \gamma \tag{3.48}$$

$$\theta = \frac{M_r}{M_{pc}} \gamma' \tag{3.49}$$

3.4.2 Fixed Base

A one-story frame and its deformed shape are shown in Fig.3-18. The same concept of analysis may be used as in section 3.3.2. From the frame in Fig.3-19, the moment distribution factors and fixed end moments are

$$D_{AB} = 0 \tag{3.50}$$

$$D_{BA} = \frac{\frac{EI_c}{h}}{\frac{EI_c}{h} + \frac{6EI_b}{L_b}} \tag{3.51}$$

$$F_{AB} = F_{BA} = -\frac{Ph}{4} \tag{3.52}$$

The tabulation of the moment distribution calculation is in Tab.3-5. According to the result, the moment diagram and the distance from the inflection point to the top of the column αh are shown in Fig.3-20. where

$$\alpha = \frac{1}{2} (1 - D_{BA}) \tag{3.53}$$

From the formula for α, it is seen that the inflection point is above the the mid height of the column. Because the fixed end has infinite stiffness, it attracts more moment than the upper end and makes the inflection point move above mid-height of the column. If $K_b >> K_c$, which makes $\alpha = \frac{1}{2}$, the column is nearly fixed at both ends, and the inflection point would be approach the middle.

By using the subassemblage model in Fig.3-15, the story can be treated as a bottom story with fixed ends. The difference is now there is no column above the subassemblage, so M_n is used instead of $2M_n$. The following equations could be established

$$M_n = - [Q (\alpha h) + P (\beta \Delta)] \tag{3.54}$$

$$M_n + M_r = 0 \tag{3.55}$$

$$\frac{\beta \Delta}{\alpha h} = \theta - \gamma \tag{3.56}$$

$$\gamma = \frac{M_r}{M_{pc}} \gamma' \tag{3.57}$$

where β is given in equation(3.45).

When using general moment-rotation charts, the value of γ' is derived with one end moment equal to zero (inflection point) and different lengths from top to the inflection point or hinged base of the column. For an interior story case, a length equal to $\frac{h}{2}$ should be used. For a hinged base, a length equal to h and for a fixed base a length equal to αh should be used.

Chapter 4
The Computer Analysis Of Subassemblage Method

4.1 The SMOA Computer Program

The computer program SMOA has been designed based on the theory of the sway subassemblage method (Armacost, 1968). A story with known member sizes is divided into subassemblages. Each subassemblage is analyzed by the computer to obtain its load-deflection relationship. A mechanism sketch for each subassemblage is in the output file. Then all subassemblages in a story are combined to gicethe load-deflection curve for the story which will also be listed in the output file.

The original program source code was written in FORTRAN IV and used in a mainframe computer. It has been revised to be executable in a personal computer and compatible for FORTRAN 77 compilers. Some changes have been made to provide the program with more functions to deal with various story conditions and to improve efficiency.

The main program and several subroutines comprise the whole program. The functions are outlined roughly as follows.

The main program
: The main program asks the user to type the input and output file names, and then reads the first line of the input file which tells how many stories are there and how many joints for each story. In the new version, it also distinguishes of the input foile is in a new or old format. The program uses a dynamic memory technique to optimize the memory used by dimensioned arrays.

Subroutine MAIN
: Subroutine MAIN repeats the calculations according to the number of floors.

Subroutine OUTIN It reads the data of member section properties, loads, frame geometry and prints the data it reads in the output file.

Subroutine SUB99 SUB99 means the subroutine executes a the DO-LOOP to the label 99 according to the direction of wind from left or right.

Subroutine SUB77 Subroutine 77 calls subroutines AINIT and AMLIM and then begins to do the calculations by subassemblages in the story.

Subroutine ZERO It sets the values for the initial situation of the subassemblage. Most of the values are zero. Then it prints those initial values.

Subroutine TRNSPS When the program is dealing with the case of the wind from the right direction, this subroutine sets all the members to an opposite sequence, so the program will work like the wind from the left.

Subroutine AINIT This subroutine calculates all the initial fixed end moments for girders and initial restraining moments for columns.

Subroutine AMLIM Subroutine AMLIM calculates the limiting moments on the ends of girders. (Driscoll et al, 1965)

Subroutine STIFF This subroutine calculates the initial stiffnesses of the girders. (Daniels, 1966a)

Subroutine AMLFT Calculates the moments on the critical locations of the left girder for the subassemblage, which are labeled as 1, X and Y from right to left.

Subroutine AMRGT Calculates the moments on the critical locations of the right girder for the subassemblage, which are labeled as 3, Y and 4 from right to left.

Subroutine POINT When a hinge is formed in subroutine SUB77, SUB77 calls subroutine POINT to find the coordinates for the load-deflection curve.

Subroutine GETQ This subroutine gets the lateral load of the subasssemblages from the sway according to different types of story.

Subroutine ARRAY Subroutine ARRAY records the coordinates from subroutine POINT and then interpolates points between two hinge points.

Subroutine ADD After all the subassemblages in a story have been analyzed, subroutine ADD adds all the load-deflection values together to obtain the load-deflection curve for the story.

Subroutine GRARR

Draw a chart of the load-deflection relationship for the story after analyzing all the subassemblages.

Subroutine SUBMEC
Draws the mechanism figure of each assemblage, shows the lengths of the members and indicates the position of hinges in the mechanism.

Subroutine BOX3 Prints the heading at the beginning of the output file.

4.2 The Input File Format

Since the program SMOA has been revised for more functions and selections, a new input data file format was designed to be used for the revised program. The old format data files are still valid for the program. SMOA can distinguish if the input file is in a new or old format and perform the same calculations. The differences between the old and new input files that are the new format has more flags to choose more selections for different situations such as top, bottom or interior stories and changes the order of input data for logic and convenience.

A sample input file is listed in Appendix.A. The format is as follows :

format	description
Card 0 : 3I5	NF,NJ,NDATA
Card 1 : 3I5	NWAY,NFLOOR,NTYPE
Card 2 : F8.2	H
Card 3 : 8F8.2	BL
Card 4 : 2F8.2	FYC,FYB
Card 5 : 9F8.0	PL
Card 6 : 9F8.0	PR
Card 7 : 8F8.2	W
Card 8 : 160	COLUMN SECTION PROPERTIES
Card 9 : 163	GIRDER SECTION PROPERTIES

NF Number of floors

NJ	Number of joints in each floor
NDATA	1 for new data type, 0 or blank for old data type
NWAY	1 for wind from left side, 2 for both sides
NFLOOR	Number of the level in the building
NTYPE	1 for top floor, 2 for interior floor, 31 for bottom floor with hinged base, 32 for bottom with fixed base, 41 for single floor with hinged base, 42 for single floor with fixed base.
H	Story height
BL	Beam spans
FYC	Yield stress of columns
FYB	Yield stress of beams
PL	Column loads when wind is from left
PR	Column loads when wind is from right
W	Beam distributed loads

The number of column and girder section properties of cards 8 and 9 depend on the number of joints. Card 8 repeats as many times as the number of NJ, and card 9 repeats NJ-1 times for each story. The section properties used here are based on punched data from Bethlehem Steel Company.

Card 1 to card 9 are repeated by the number NF. The meanings of symbols used in the program are also listed in nomenclature chapter which includes all the important symbol meanings in program SMOA.

Additional items of numerical data serving as labels may be at the end of cards 4, 5 and 6 in the format of (I2,I3). The first number is the level no. That tells which floor is represented in the building. The second number is either 24, 25 or 27 which represents W, PL or PR. These numbers can be created by some design programs as an output file which can be used as an input file for SMOA.

4.3 The Direct Method

The original SMOA computer program written for the subassemblage analysis to get the load-deflecton relationship for structures used a small constant increment to reach the results. The rotation θ of the upper end of the column was increased by a small amount which is determined by the user to enter a small value in the program. The rotation angle would start from zero and increase by the increment value. Every time the increment was added to the rotation angle, the stresses in the entire subassemblage would change and be calculated by computer. The program will check if the stress at any place in the subassemblage reaches a yielding condition, then a hinge is formed at that location. The program would stop increasing increments when a mechanism is formed. The time needed for processing this procedure is determined by the value of increments chosen by the user. A large value of increment would diminish the time for executing the program, but less accuracy would result when the increment is larger. A rather long time would be needed for a small increment such as 0.00001 to be executed on a PC in order to get a satisfactory result (Armacost, 1968).

A study (Driscoll, 1975) shows that it is not necessary to increase by such a small amount and calculate the stresses of the members every time. A manual method has shown that the next increment of rotation at the upper end of the column can be found for the next yield location in of the sub-assemblage. All the tentative increments of rotation of the joint could be calculated to cause each of the critical places to form a hinge. These are the ends of the girders, the upper end of the column, and points of zero shear under distributed load. The smallest tentative increment is chosen as the increment of rotation for the subassemblage to form the next hinge directly. This method spends less time than the original procedure and moves directly to the answers.

The revised SMOA program has included this feature. It compares the increments it needs to form a hinge at two ends of girders and the upper end of the column. One thing that should be noticed is that if wind comes from the left, the moments at both of the right and left ends of girders would increase clockwise. When the moment of the left end changes from countclockwise to clockwise, it probably is not the largest moment in the girder other than the moment at the right end. The maximum moment could happen at the interior portion of the girder. A calculation is made to determine the point of zero shear which has already been handled by SMOA. Those locations with maximum moments in the interior part of girders are called X when formed at right girders and Y when formed at girders left of a column.

Chapter 5
Example Of Computer Analysis By SMOA

5.1 The Sample Frame

In this chapter, a frame will be presented as a sample to show how the program SMOA works.

The frame type I in reference (Lu, 1975) will be used as shown in Fig.5-1. The member sizes are based on design II in that report which chooses the in-plane direction column length factor K by using the AISC alignment chart and assume the out-of-plane K to be unity(Ky=1.0). The member sizes chosen for this design are

Level	Beams	Columns
1	W12X22	W8X24
2	W14X22	W8X18
3	W14X22	W8X35
4	W14X22	W8X31
5	W14X26	W8X48
6	W14X26	W8X48
7	W14X30	W8X58
8	W14X30	W8X58
9	W16X31	W8X67
10	W16X31	W8X67

The load conditions of the frame are

Roof load		
	Live load	30 psf
	Dead load	40 psf
Floor load		
	Live load	40 psf
	Dead load	55 psf
Wall		9.5 kips
Wind load		20 psf

and the bent spacing is 20 ft.

The results will be compared with another computer program "UBFR" which also considers the P-Δ effect. The output shows all the force distributions and deflections in each member of the frame. Here we choose the top floor, the fifth floor and the bottom floor and plot their free body diagrams in Fig.5-2.

5.2 SMOA Analysis

The input file for generating the member section properties and loads for analyzing the top, the fifth and the bottom floors by SMOA is in Appendix. B. The column loads came from the output of UBFR. The distributed loads can be calculated by the data in previous section. Because this is a symmetrical building, it is only analyzed for wind coming from one direction. According to such boundary condition, different flags are given in the input file.

By executing SMOA, three load-deflection curves for the three floors from the output of SMOA are in Figs. 5-3, 5-4 and 5-5. These three charts can be used to check whether the preliminary design is reasonable. From the charts, the ultimate shear force for the top story is about 48.K, the fifth story is 52k, and 50k for the bottom story. Referring to the previous section, the wind load is 20 pounds per square foot. The shear force at the bottom floor can be calculated as $20 \cdot 20 \cdot 9.5 \cdot 10 = 38000$, which is 38.K and less than 50.K. The shear force of the top and the fifth story would be about one tenth and half of the bottom story and the ultimate shear force is about the same as the bottom story. Comparing the numbers, the shear force of the top story is much less than its ultimate shear force, so the upper part of the building is controlled by the gravity load. The shear force of the bottom story is near to its ultimate shear force. The lateral load controls the lower part of the building. The middle part of the building is a transition zone which should be considered for both of the factors. In this case, we can see this design is reasonable.

Chapter 6
Summary

The sway subassemblage method is based on the concept of sway subassemblages. A story can be divided into several sway subassemblages. Each contains a restrained column and one or two restraining girders. The method considers the reduction of strength due to P-Δ effects and also the plastification of the columns and beams and is able to determine the approximate lateral load versus sway deflection curve of a story.

The analysis discussed in this thesis enables the sway subassemblage method to deal not only with the interior stories but also top and bottom stories. An assumption has been made when dealing with interior stories in a frame by the sway assemblage method that the inflection point is at mid-height of the column. In dealing with top or bottom stories, some other analysis has to be made. Especially for top story and fixed end bottom story subassemblages, a cantilever moment distribution method is used in analysis of the subassemblages. A special case of a one-floor frame is also discussed here.

The procedure for creating a load-deflection curve for combined load analysis by the subassemblage method can be made using special prepared design charts (Parikh, 1965). The procedures of this method also have been put into a FORTRAN computer program "SMOA". By providing input data including frame type, member size and load conditions, the user can obtain an output file which gives the load-deflection curve by stories and the sequence of formation of the plastic hinges in a story.

Tables

	AB	BA
DF		$\frac{1}{7}$
FEM	$-\frac{P\ L}{4}$	$-\frac{P\ L}{4}$
		$\frac{P\ L}{28}$
CO	$-\frac{P\ L}{28}$	
moment	$-\frac{2}{7}\ P\ L$	$-\frac{3}{14}\ P\ L$

Table 3-1: An Example of Moment Distribution Tabulation

	AB	BA	BC	CB	CD	DC
DF		$\frac{1}{14}$	$\frac{1}{14}$	$\frac{1}{14}$	$\frac{1}{14}$	$\frac{1}{7}$
FEM	-45	-45	-27	-27	-9	-9
		5.14	5.14	2.57	2.57	1.29
CO	-5.14		-2.57	-5.14	-1.29	-2.57
		0.184	0.184	0.46	0.46	0.26
COLUMN	-50.14	-39.68	-24.25	-29.11	-7.26	-10.02
BEAM		63.93		36.37		10.02

Table 3-2: Moment Distribution Tabulation of a Three Story Building

	other part below the top story	AD	AB	BA
DF	⋮	D_{AD}	D_{AB}	D_{BA}
FEM		$-\frac{3}{4}Ph$	$-\frac{1}{4}Ph$	$-\frac{1}{4}Ph$
		$D_{AD}\,Ph$	$D_{AB}\,Ph$	$\frac{1}{4}D_{BA}\,Ph$
CO			$-\frac{1}{4}D_{BA}\,Ph$	$-D_{AB}\,Ph$
moment			$(-\frac{1}{4}+D_{AB}-\frac{1}{4}D_{BA})\,Ph$	$(-\frac{1}{4}+\frac{1}{4}D_{BA}-D_{AB})\,Ph$

Table 3-3: Moment Distribution of Top Story Subassemblage

	AB	BA	BC	other parts above the bottom story
DF		D_{BA}	D_{BA}	
FEM	$-\frac{1}{4}(n-\frac{1}{2})\,Ph$	$-\frac{1}{4}(n-\frac{1}{2})\,Ph$	$-\frac{1}{4}(n-\frac{3}{2})\,Ph$	
		$\frac{1}{2}(n-1)\,D_{BA}\,Ph$	$\frac{1}{2}(n-1)\,D_{BA}\,Ph$	
CO	$-\frac{1}{2}(n-1)\,D_{BA}\,Ph$			
moment	$-\frac{1}{4}(n-\frac{1}{2})\,Ph$ $-\frac{1}{2}(n-1)\,D_{BA}\,Ph$	$-\frac{1}{4}(n-\frac{1}{2})\,Ph$ $+\frac{1}{2}(n-1)\,D_{BA}\,Ph$		

Table 3-4: Moment Distribution of Bottom Story Subassemblage with Fixed Base

	AB	BA
DF		D_{AB}
FEM	$-\frac{Ph}{4}$	$-\frac{Ph}{4}$
		$\frac{D_{BA}}{4}Ph$
CO	$-\frac{D_{BA}}{4}Ph$	
MOMENT	$-(\frac{1}{4}+\frac{D_{BA}}{4})\,Ph$	$-(\frac{1}{4}-\frac{D_{BA}}{4})\,Ph$

Table 3-5: Moment Distribution of Single Story Subassemblage with Fixed Base

Figures

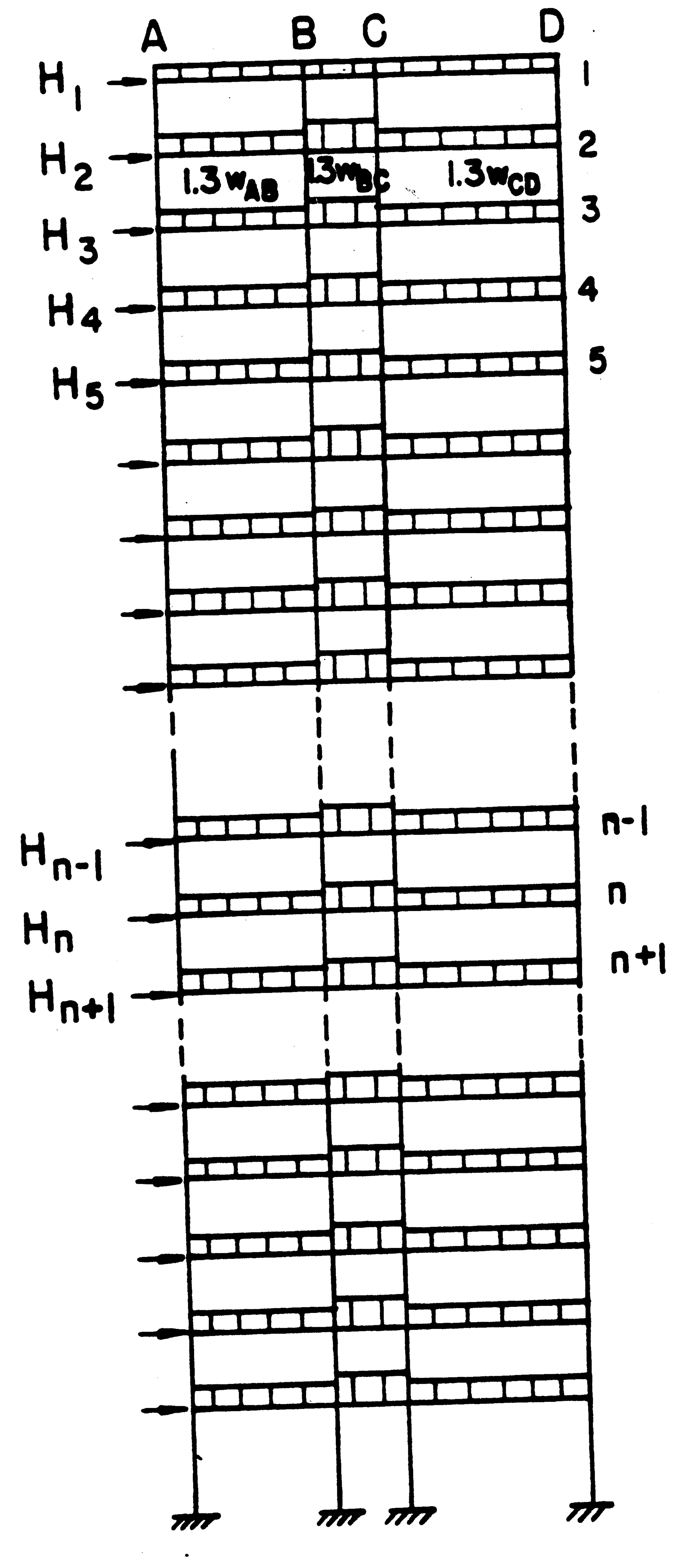


Figure 2-1: Sample Frame

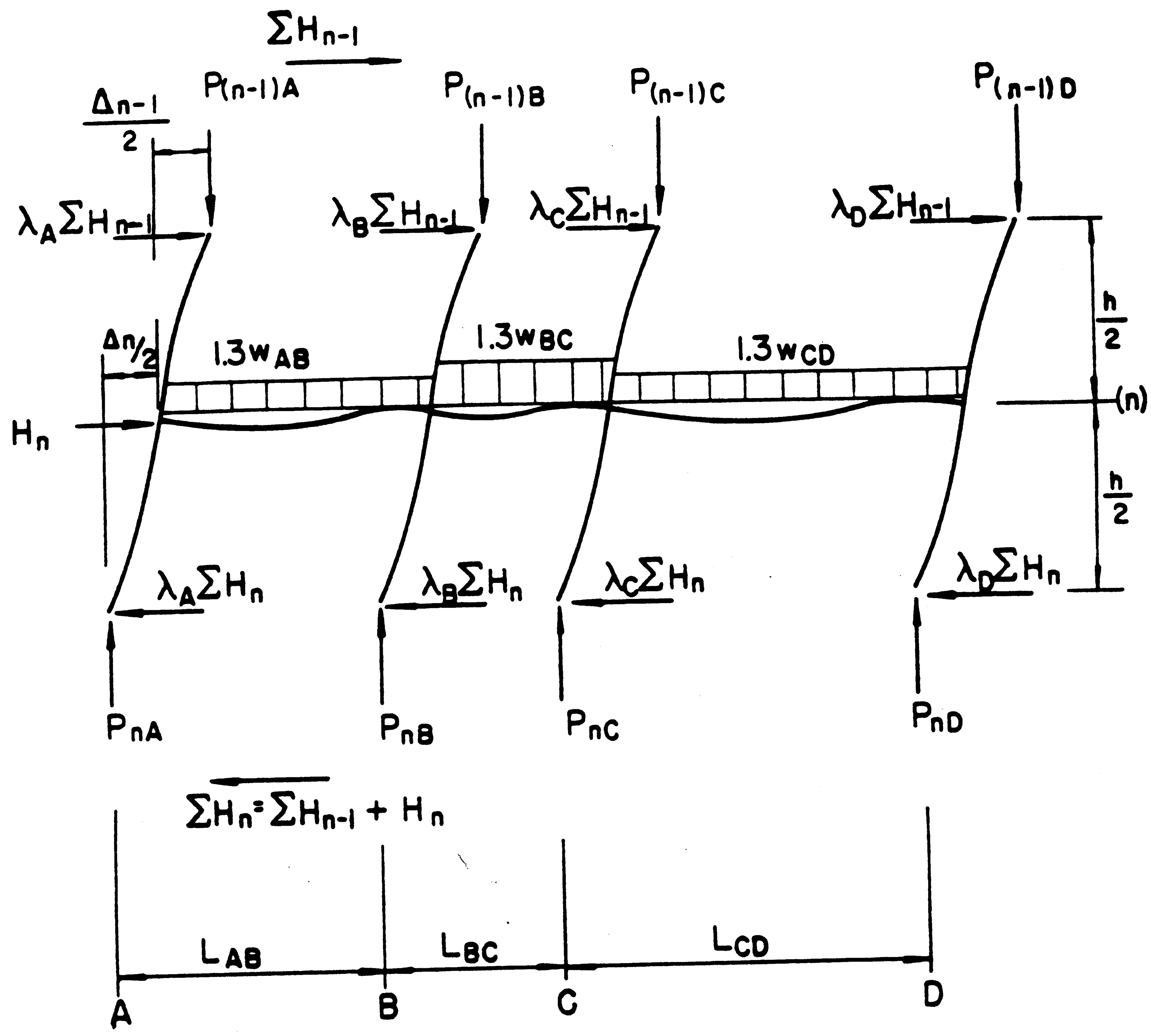


Figure 2-2: Central Portion of A Two-Story Unit

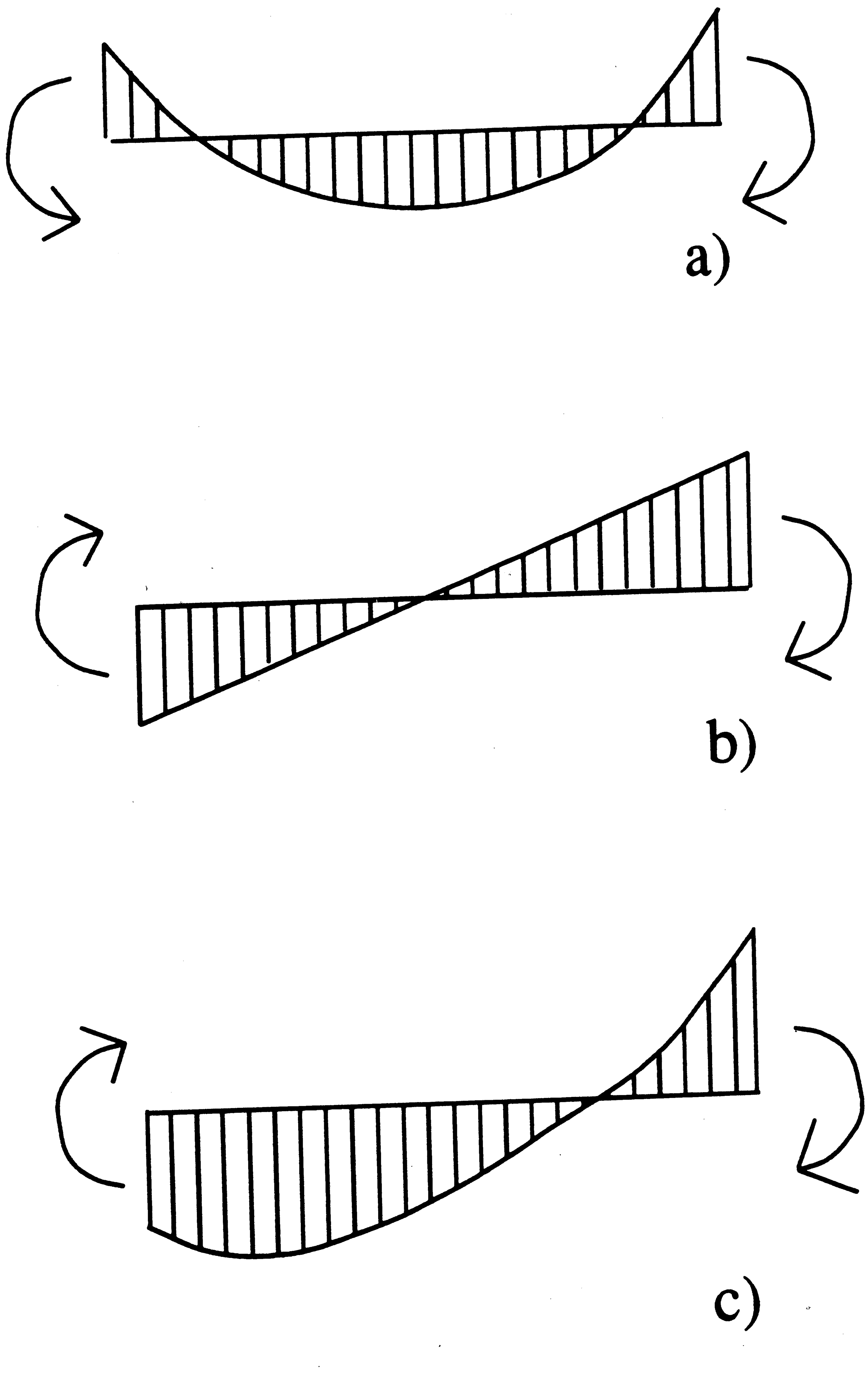


Figure 2-3: Moment Diagrams of Girder under a. Gravity Load; b. Wind load; c. Combined Load

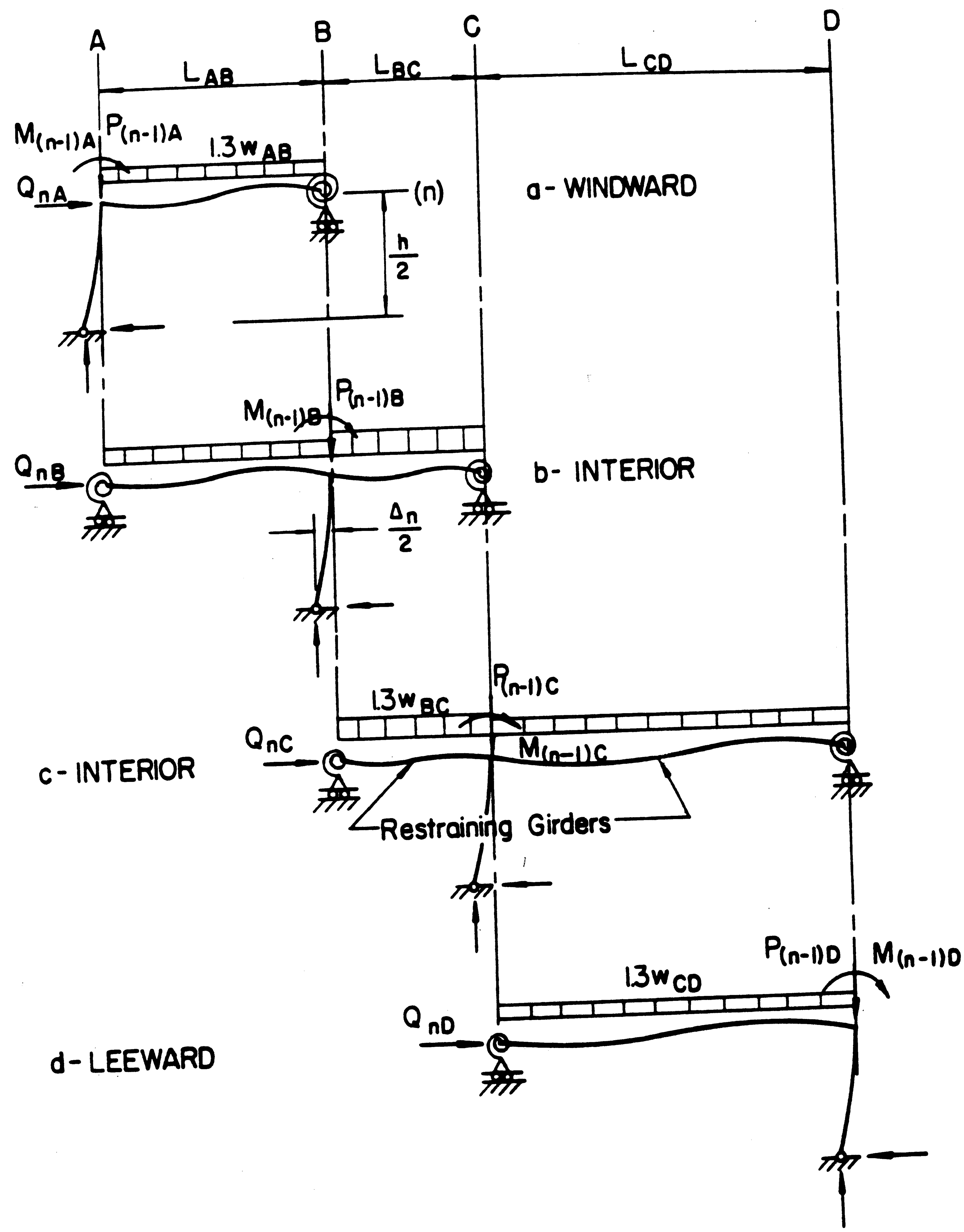


Figure 2-4: Sway Subassemblages At Level n

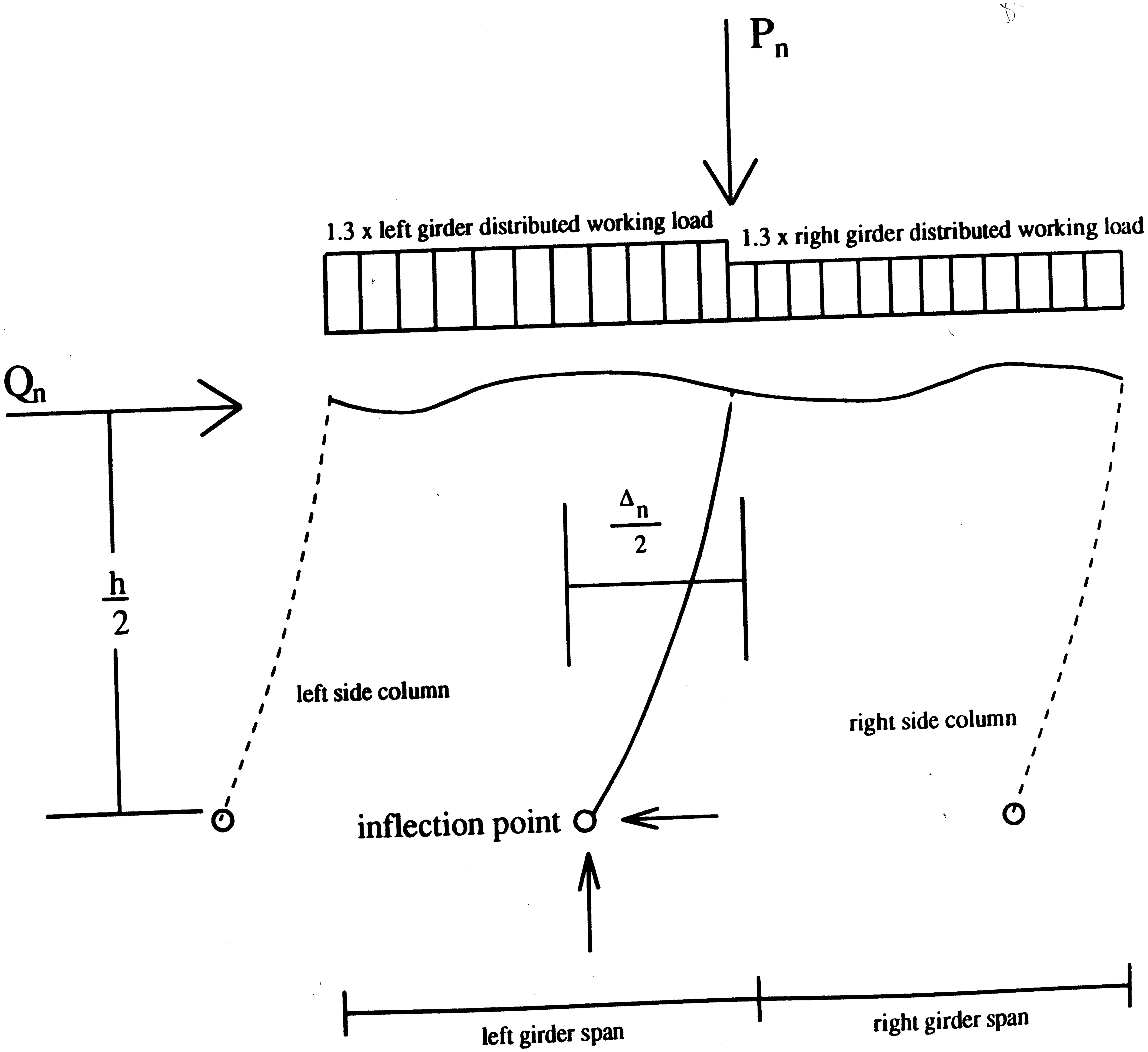


Figure 2-5: A General Subassemblage

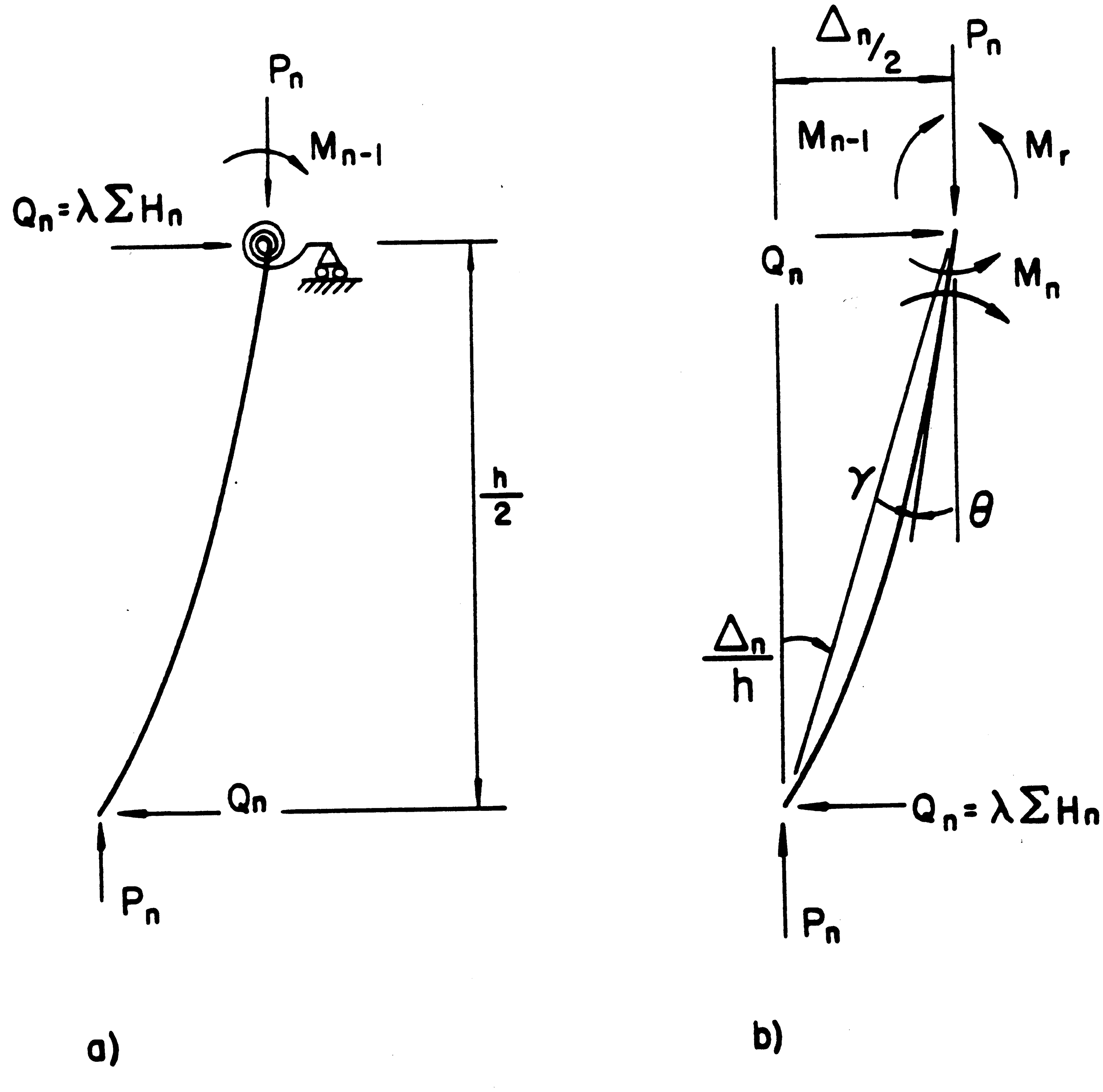


Figure 2-6: Sway Subassemblage Analysis Model

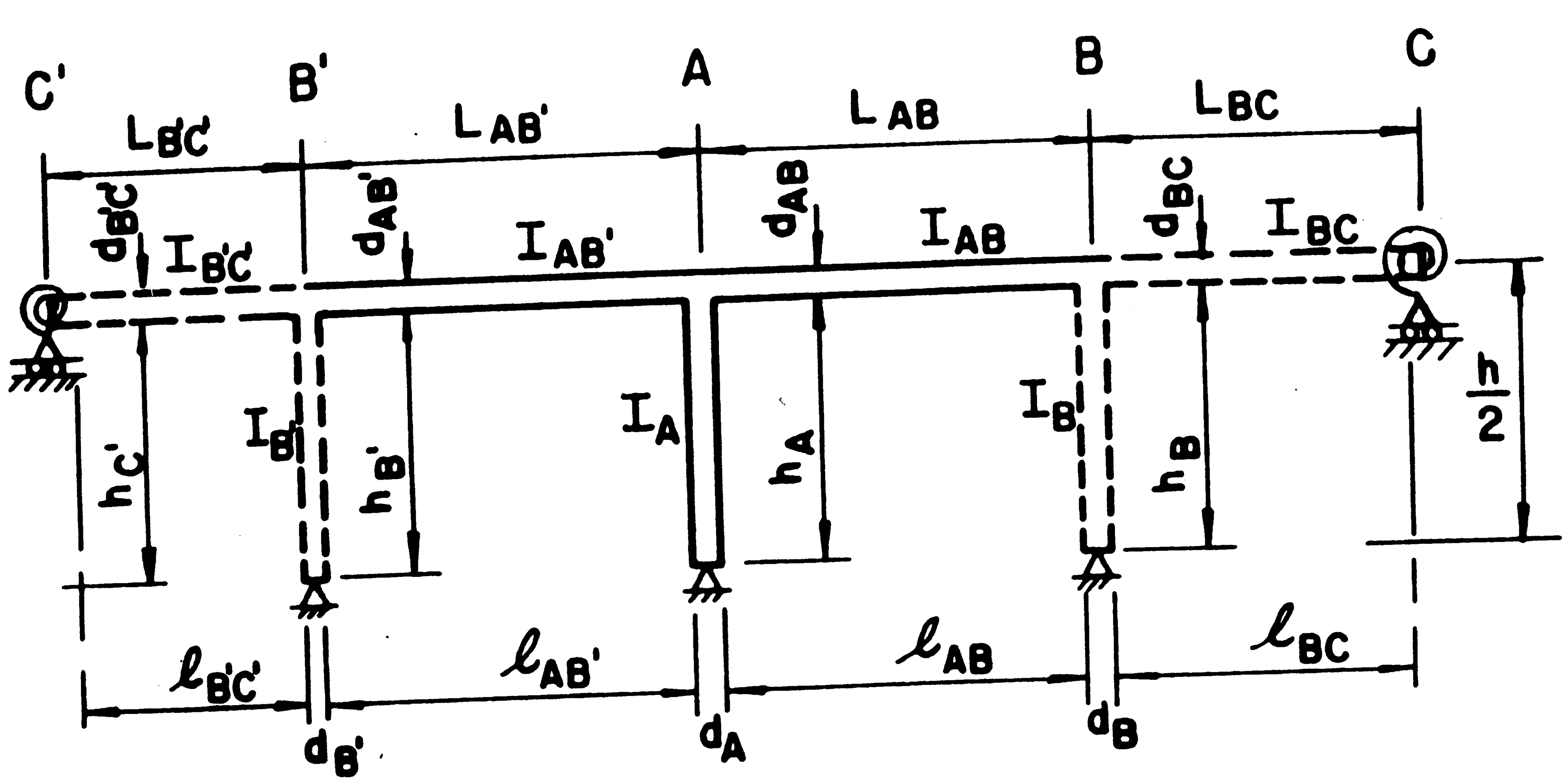


Figure 2-7: Frame For Determining Initial Restraining Coefficients at A

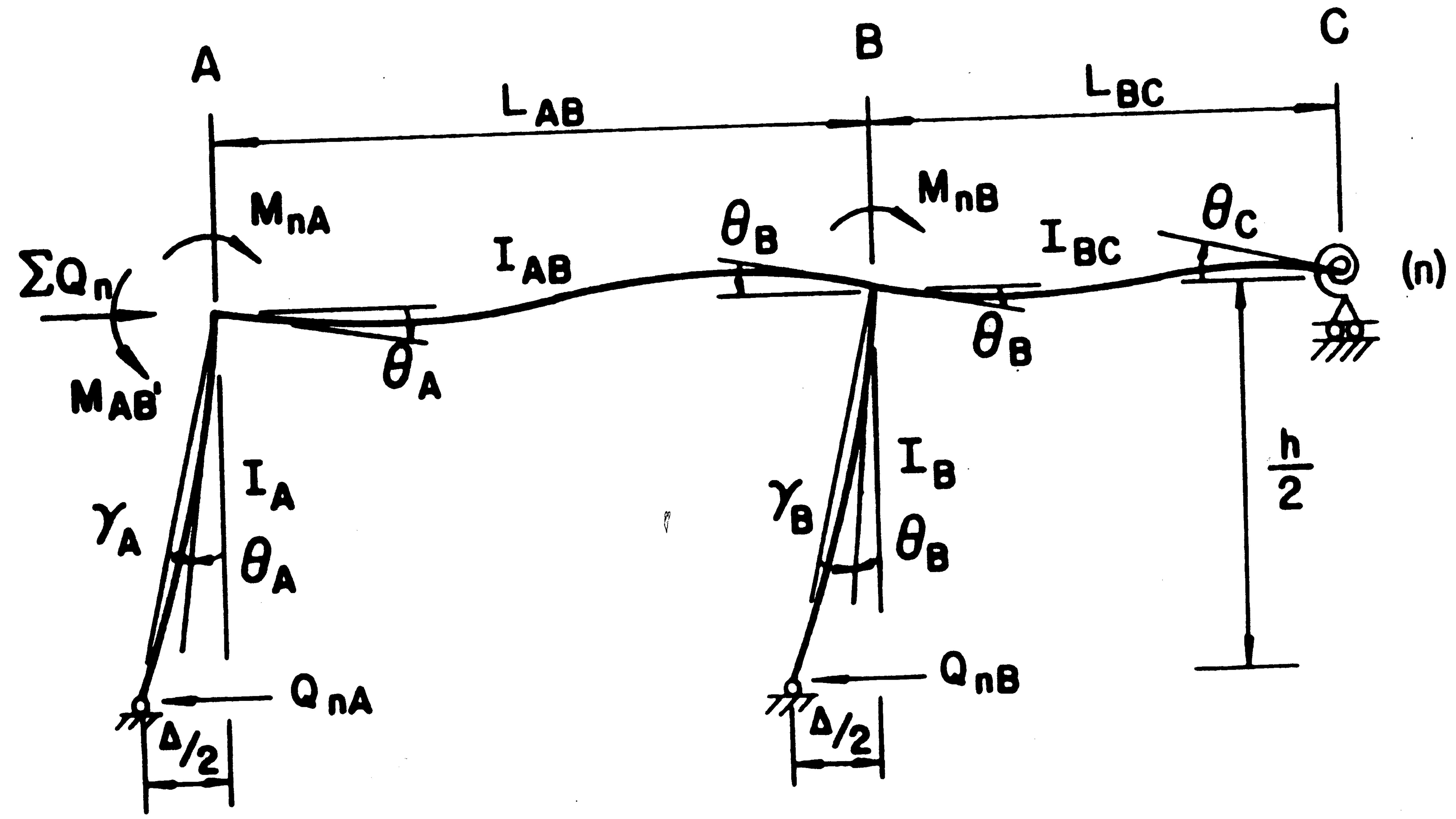


Figure 2-8: Frame for Determining K_{AB}

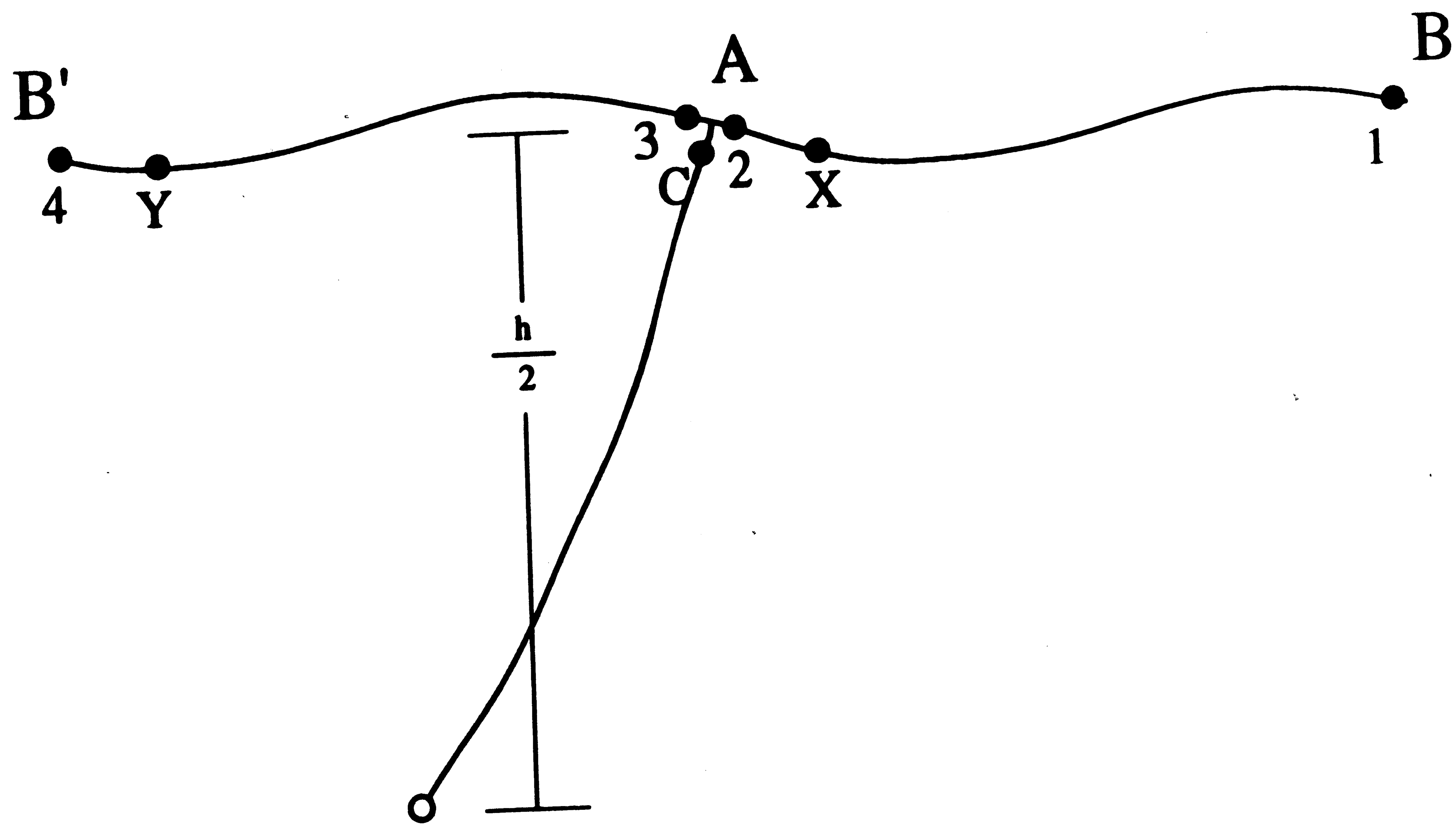


Figure 2-9: The Critical Yield Locations in a Subassemblage

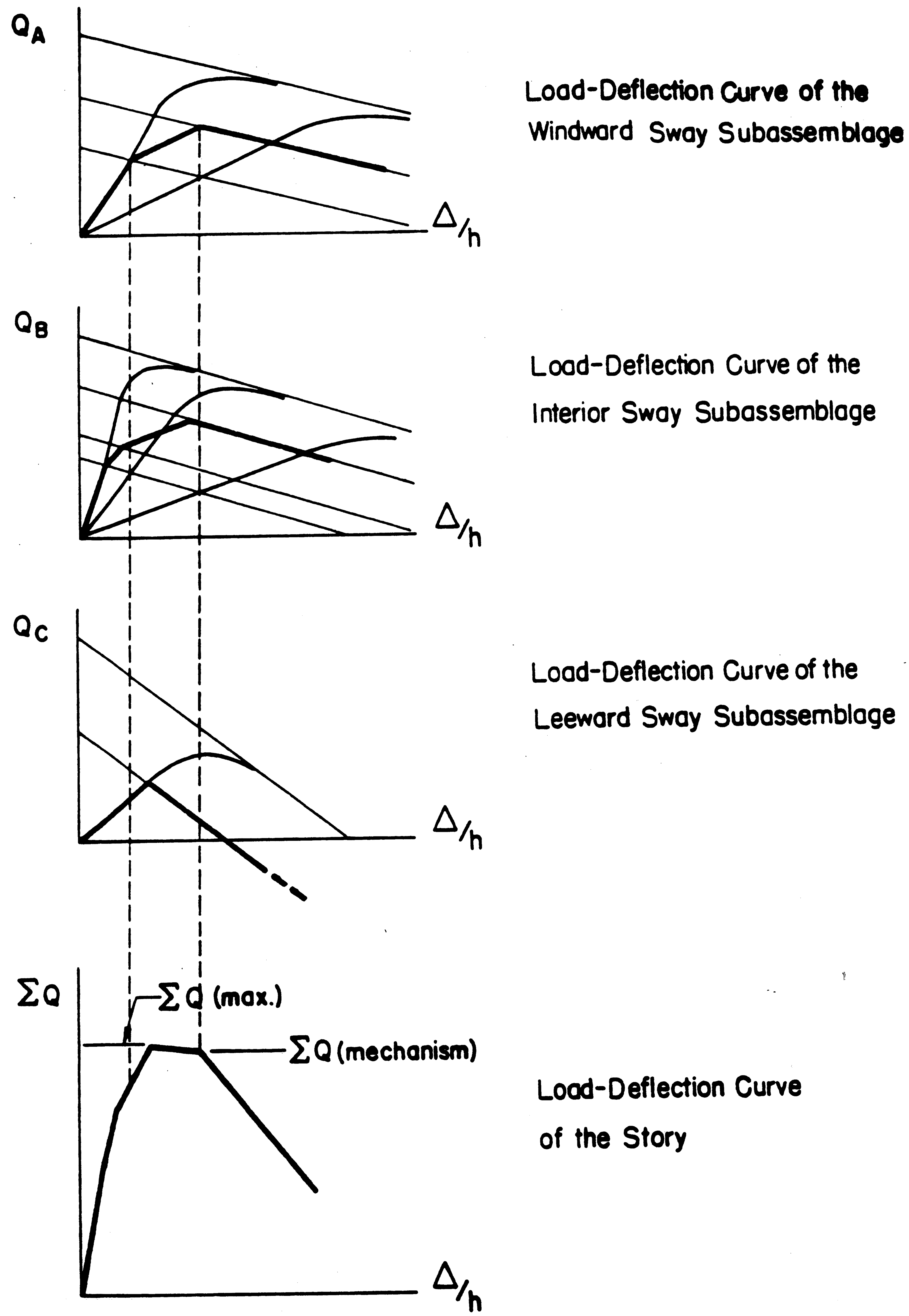


Figure 2-10: Subassemblage and Story Load-Deflection Curves

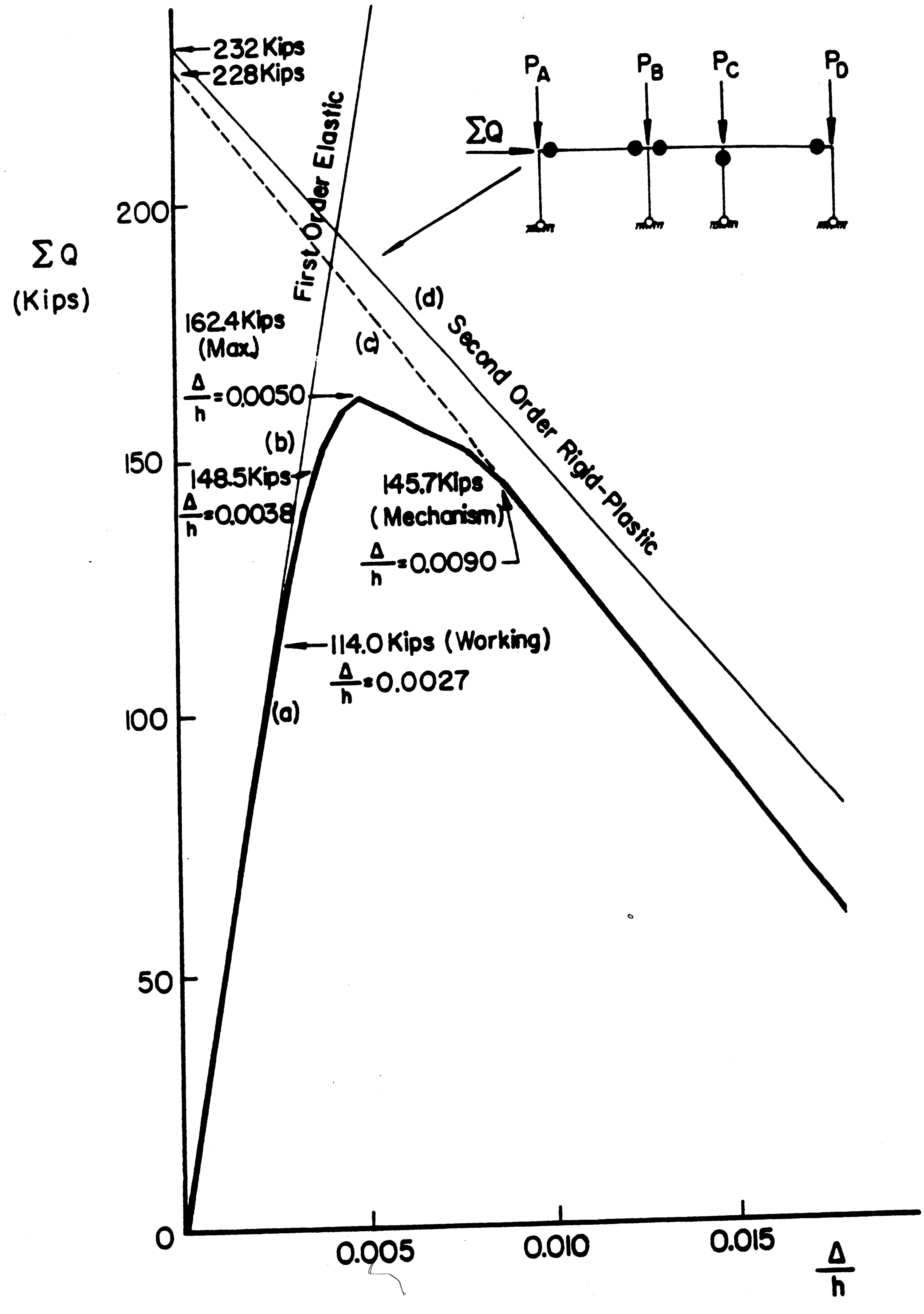


Figure 2-11: A Sample Load-Deflection Curve

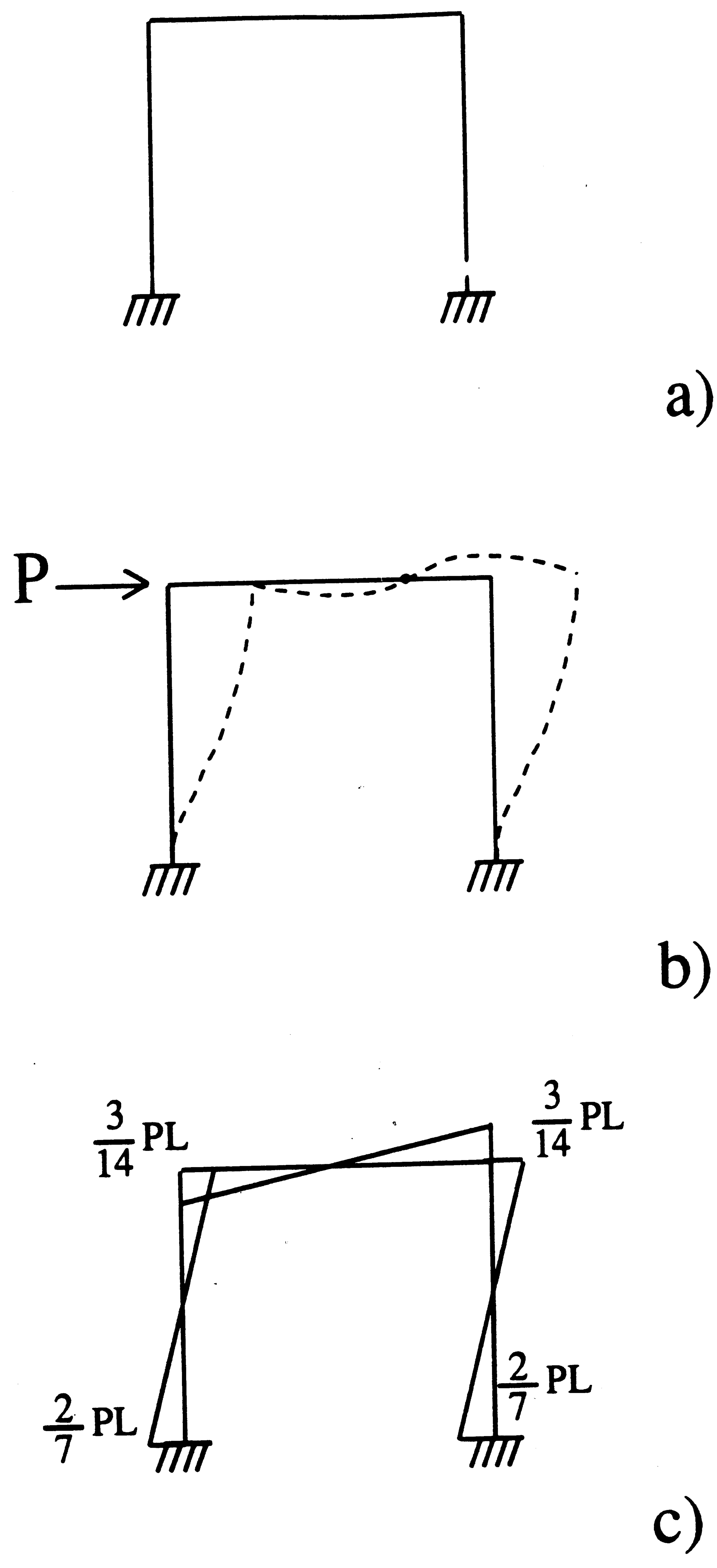


Figure 3-1: A Single-Story, Single-Span Frame

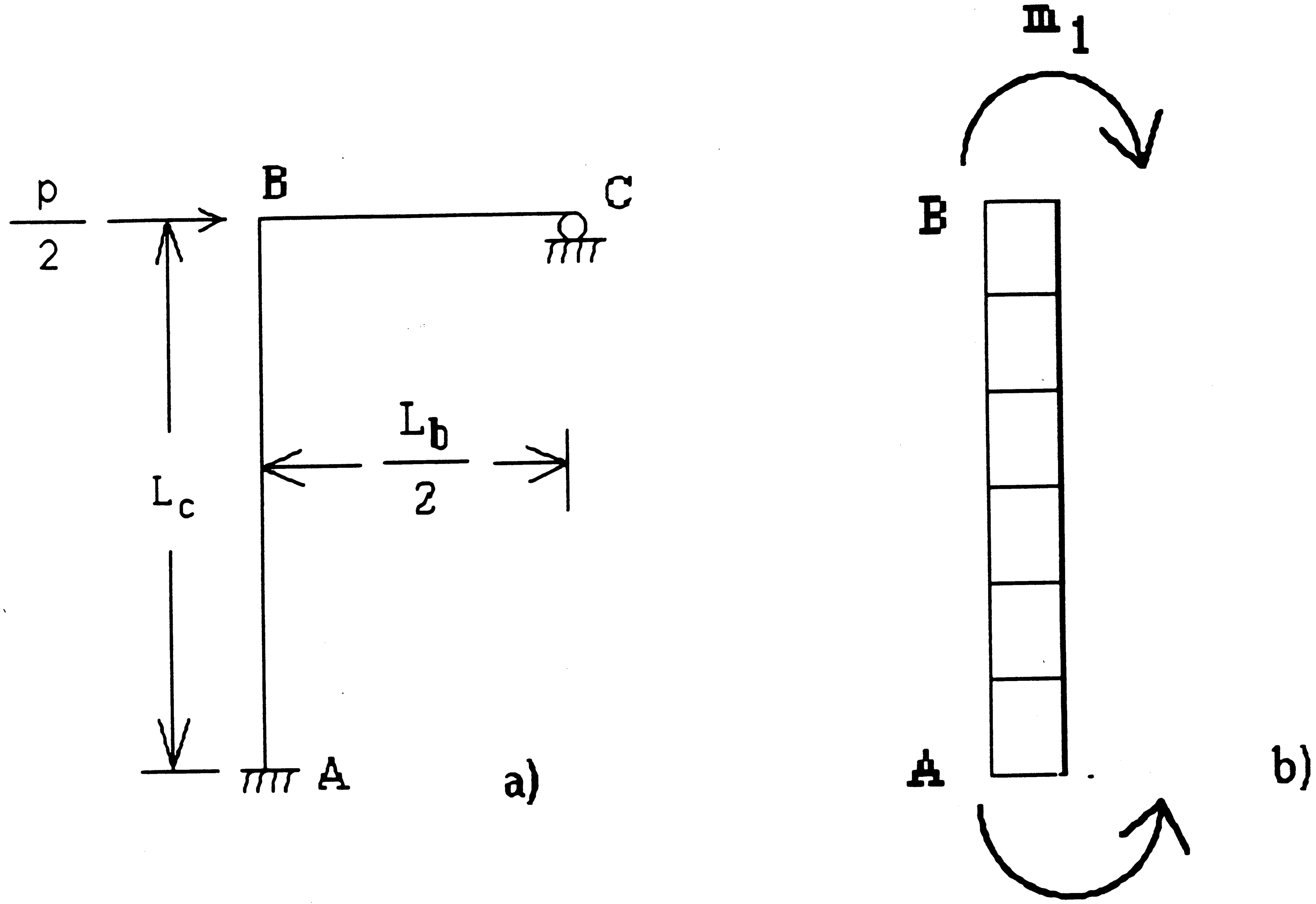


Figure 3-2: Cantilever Moment Distribution Method Model

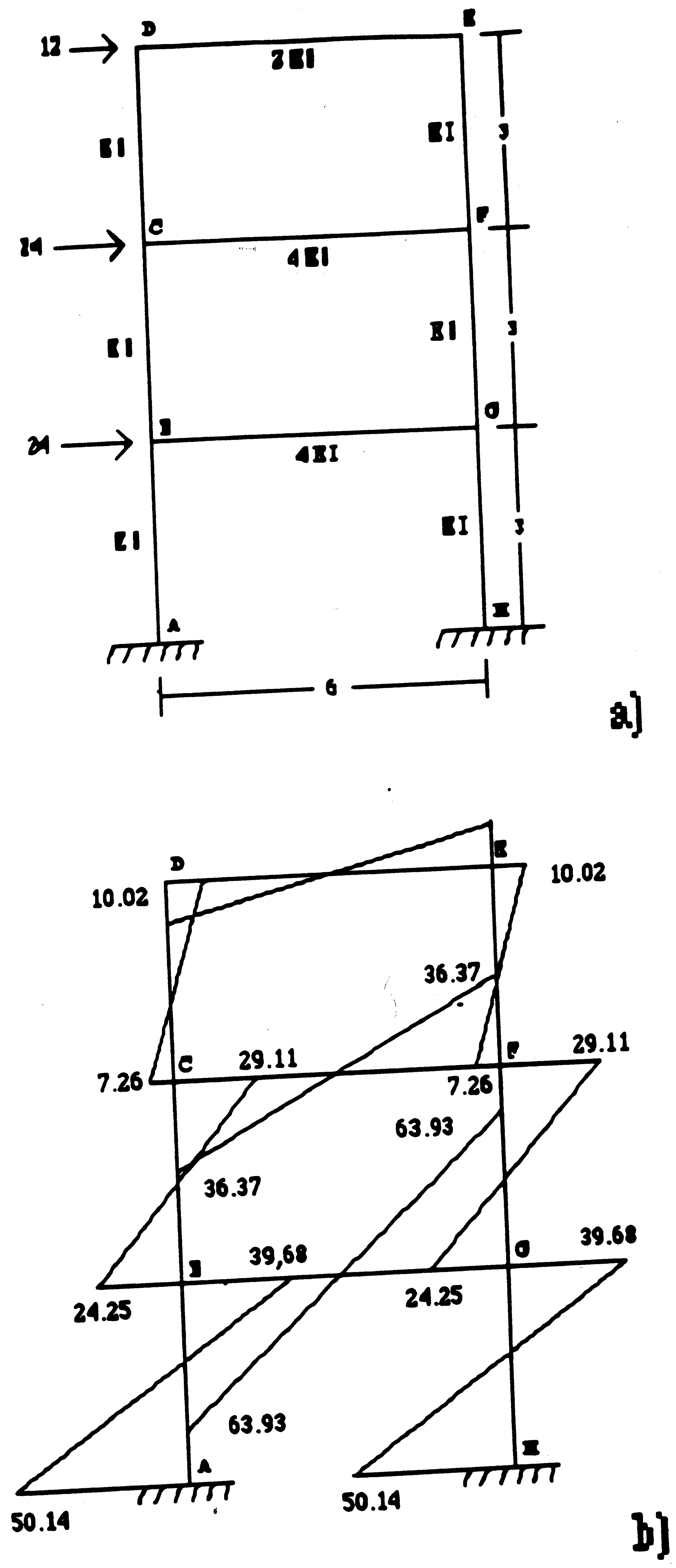


Figure 3-3: A Three-Story, Single Span Sample Frame

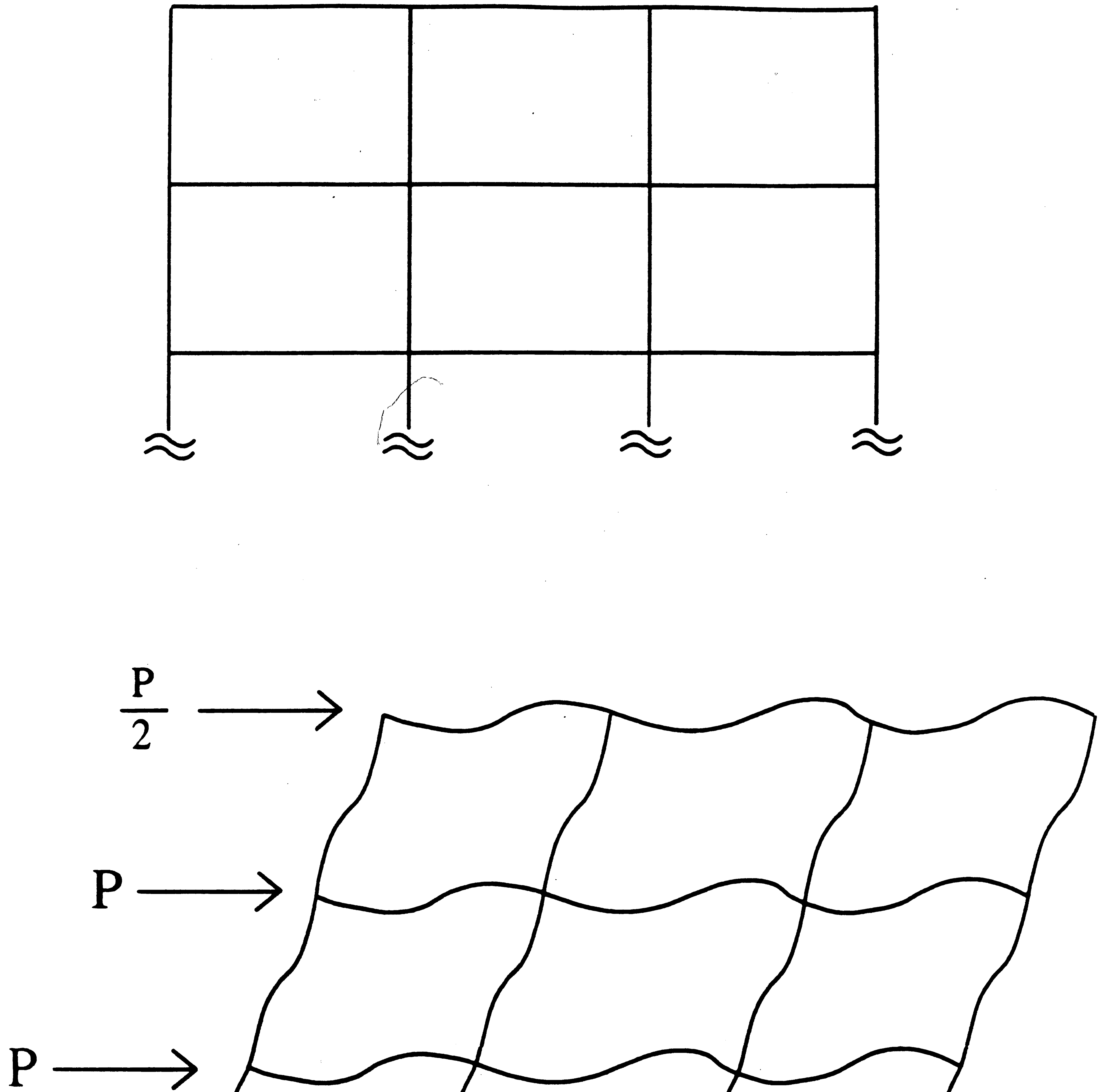


Figure 3-4: Upper Part of a Tall Building

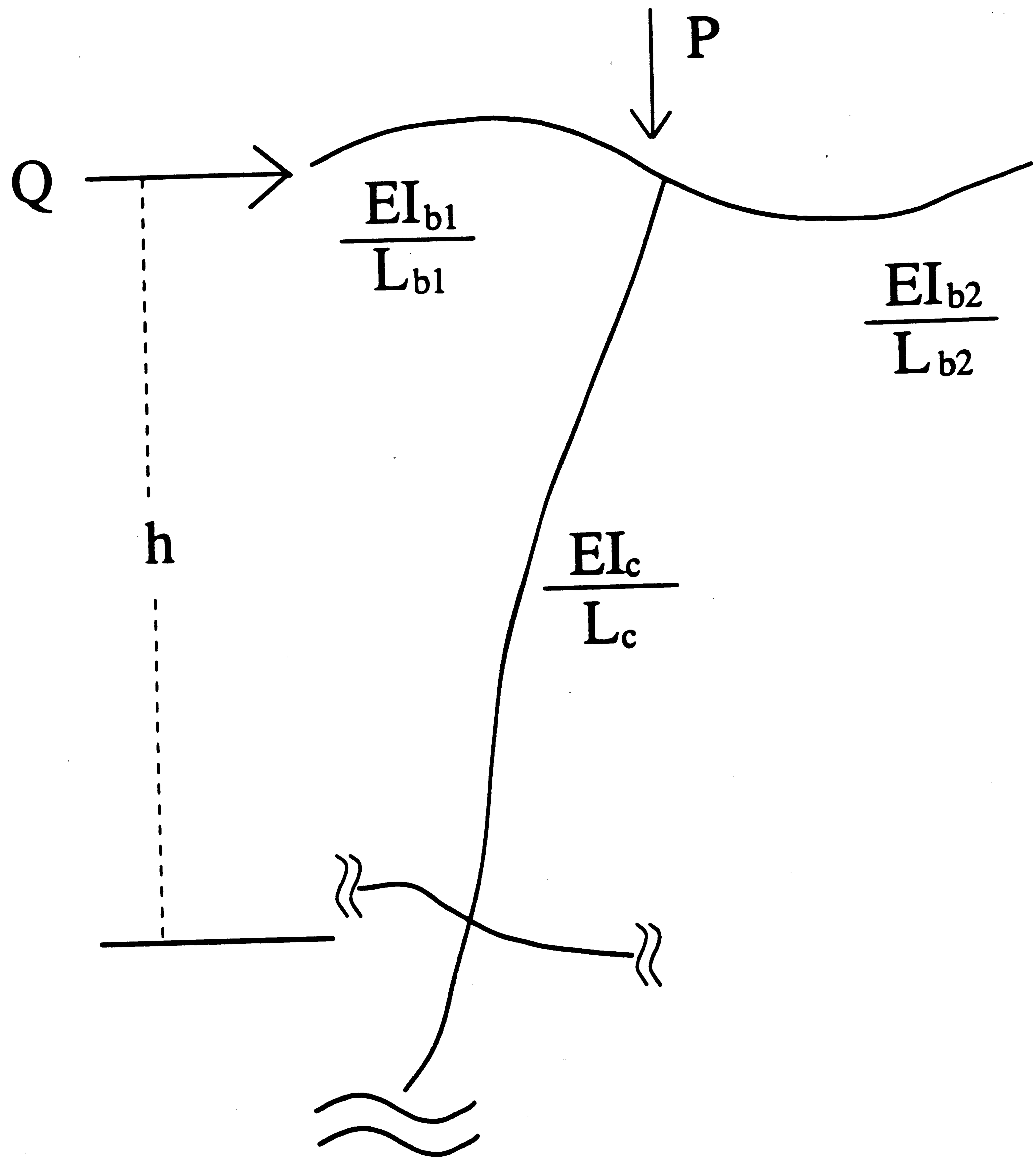


Figure 3-5: A General Subassemblage of Top Story

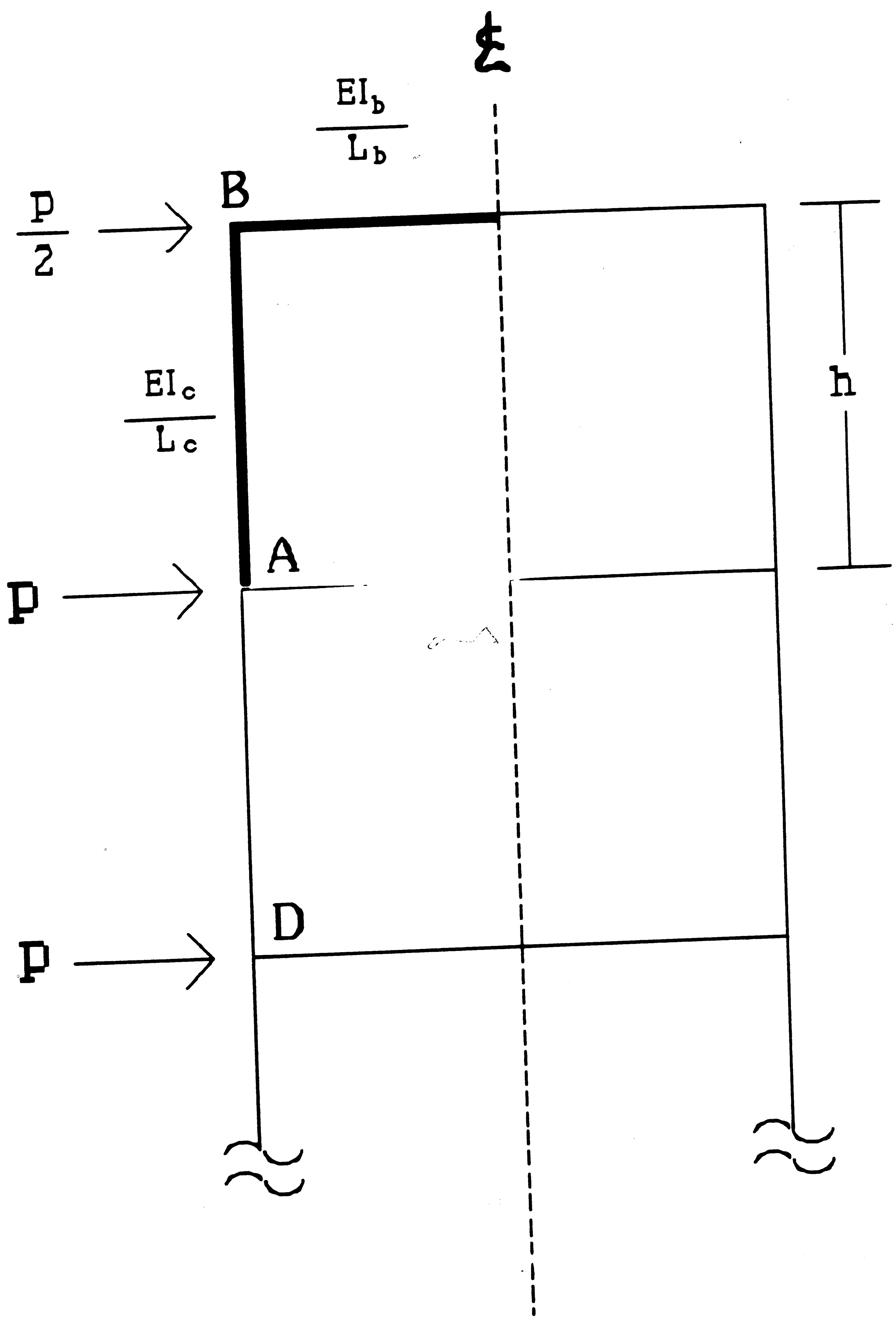


Figure 3-6: Top Story Subassemblage Moment Distribution Method Model

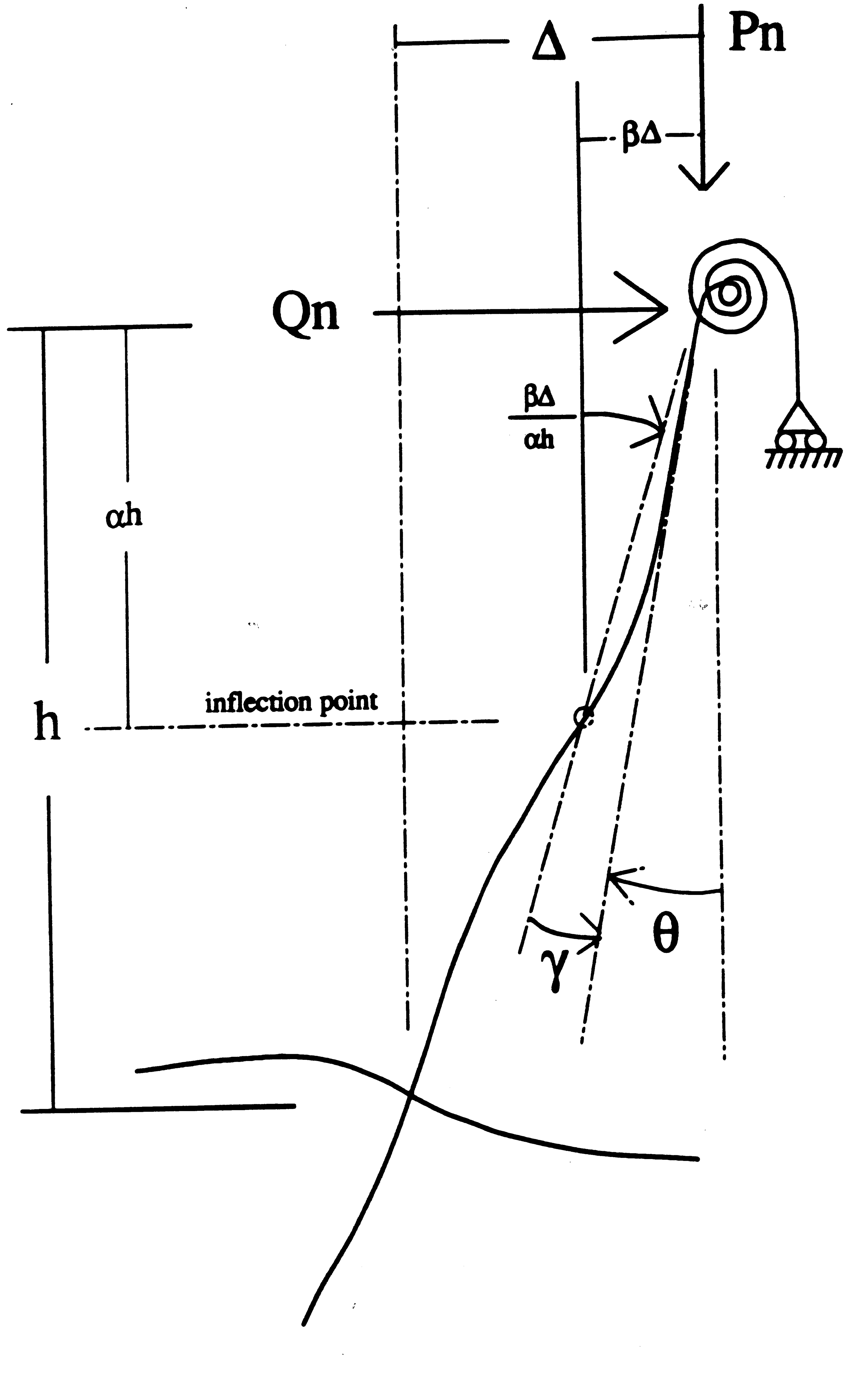
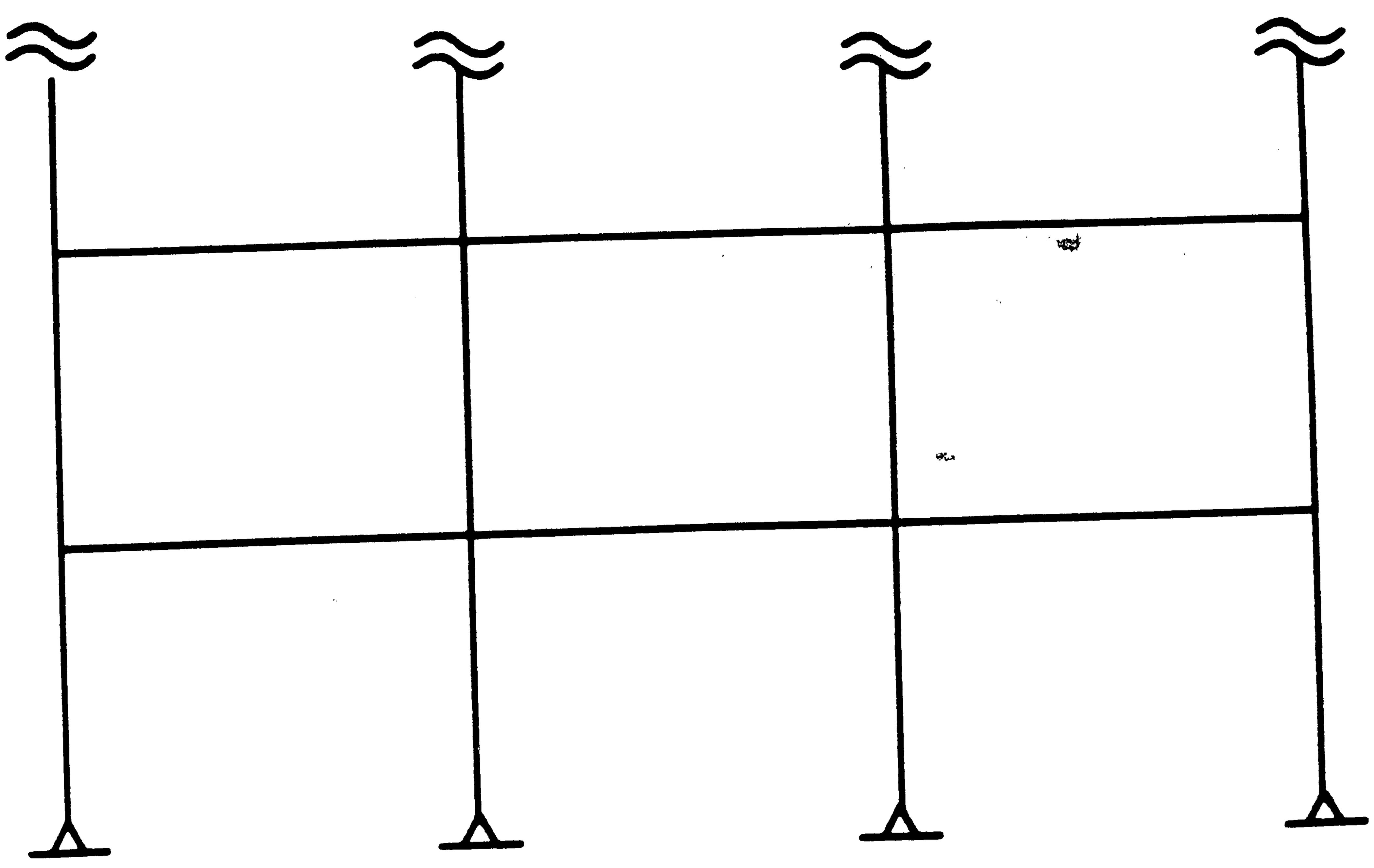


Figure 3-7: Subassemblage Model for Top Story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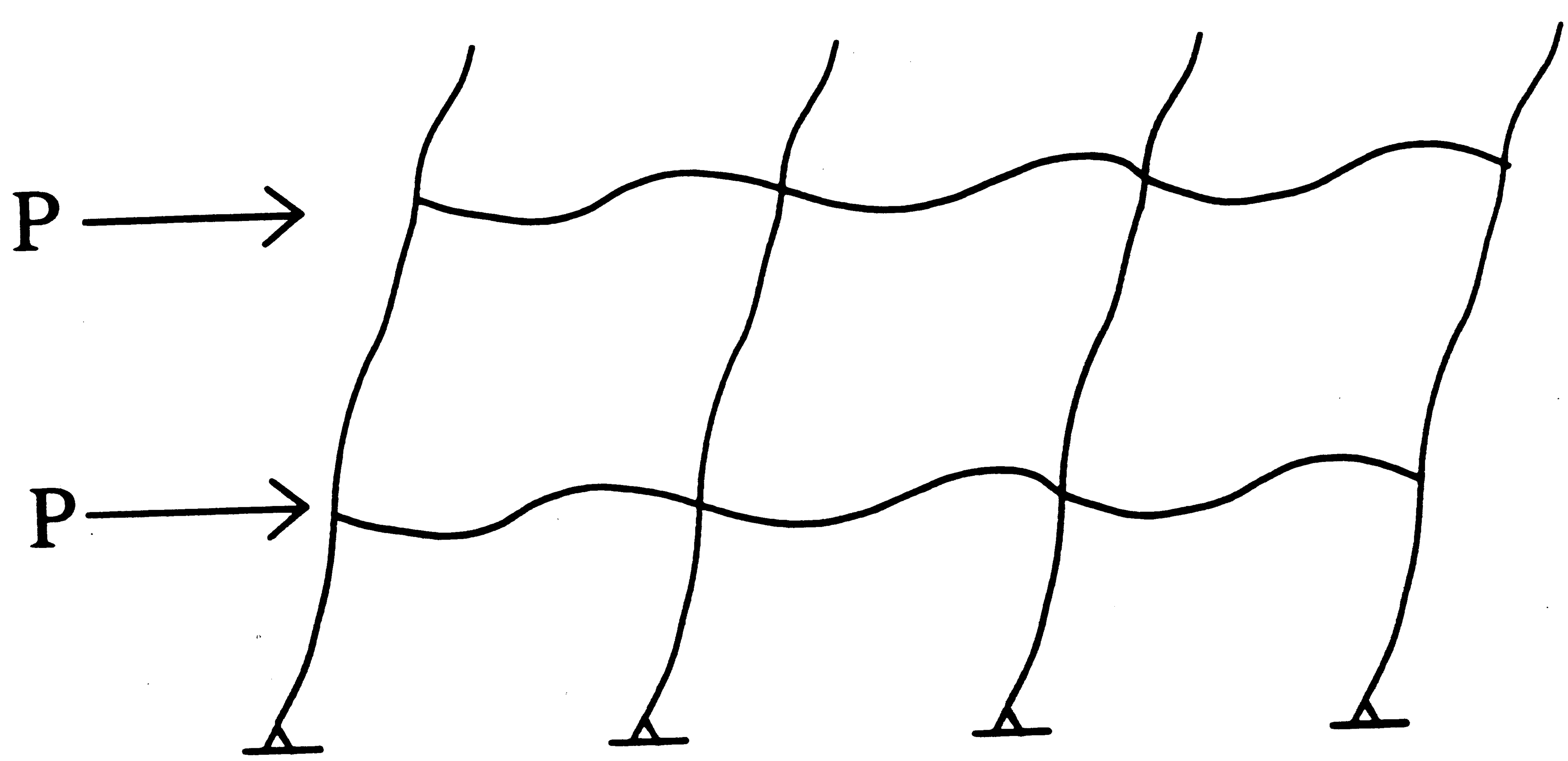


b)

Figure 3-8: Lower Part of a Building With Hinged Base

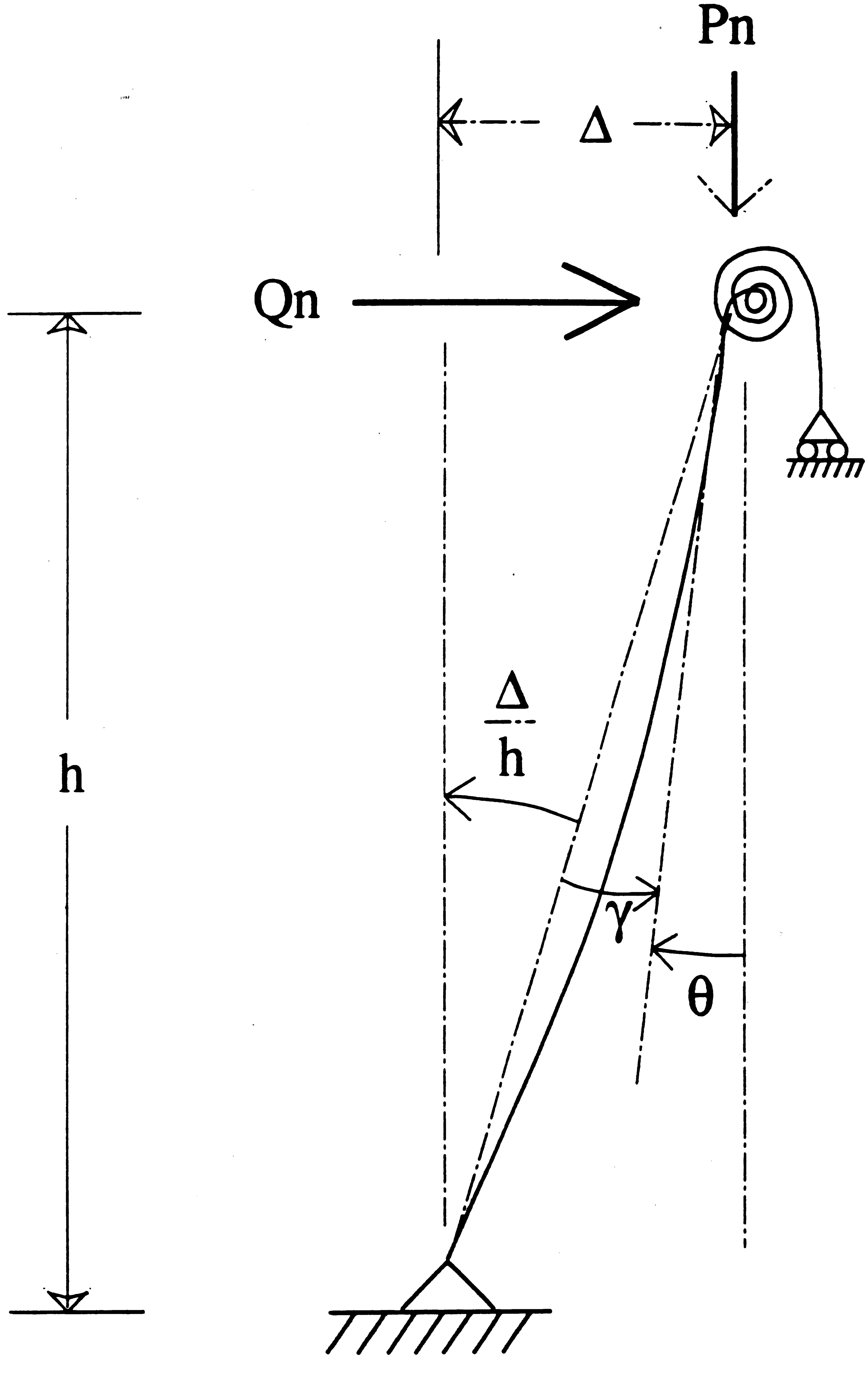
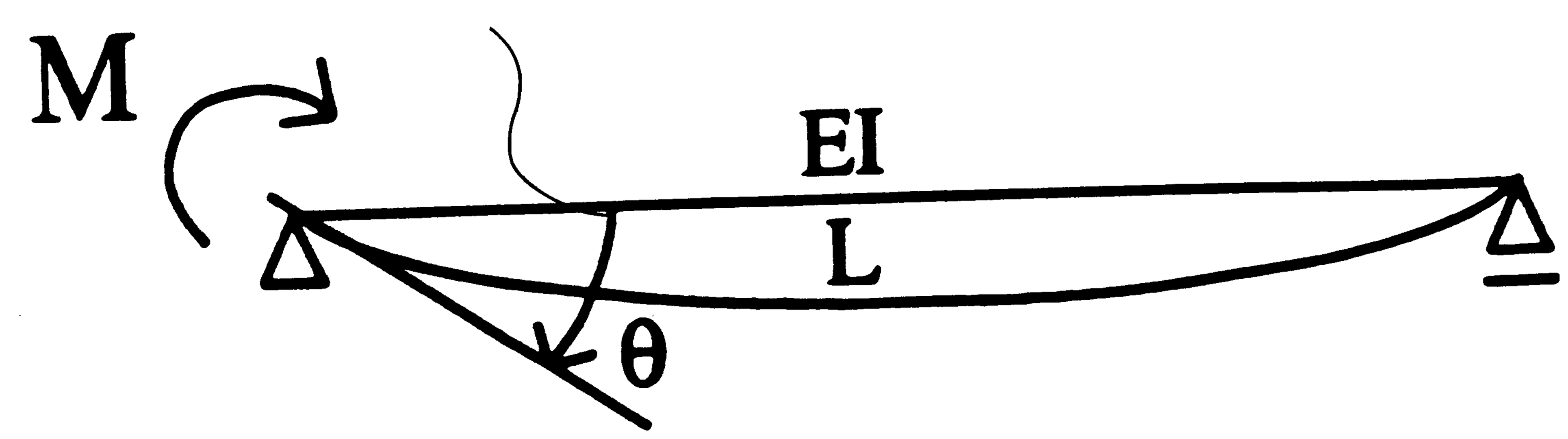


Figure 3-9: Subassemblage Model for Bottom Story With Hinged Base



$$M = \frac{3EI}{L}\ \theta$$

Figure 3-10: A Simply Supported Beam with Moment at One End

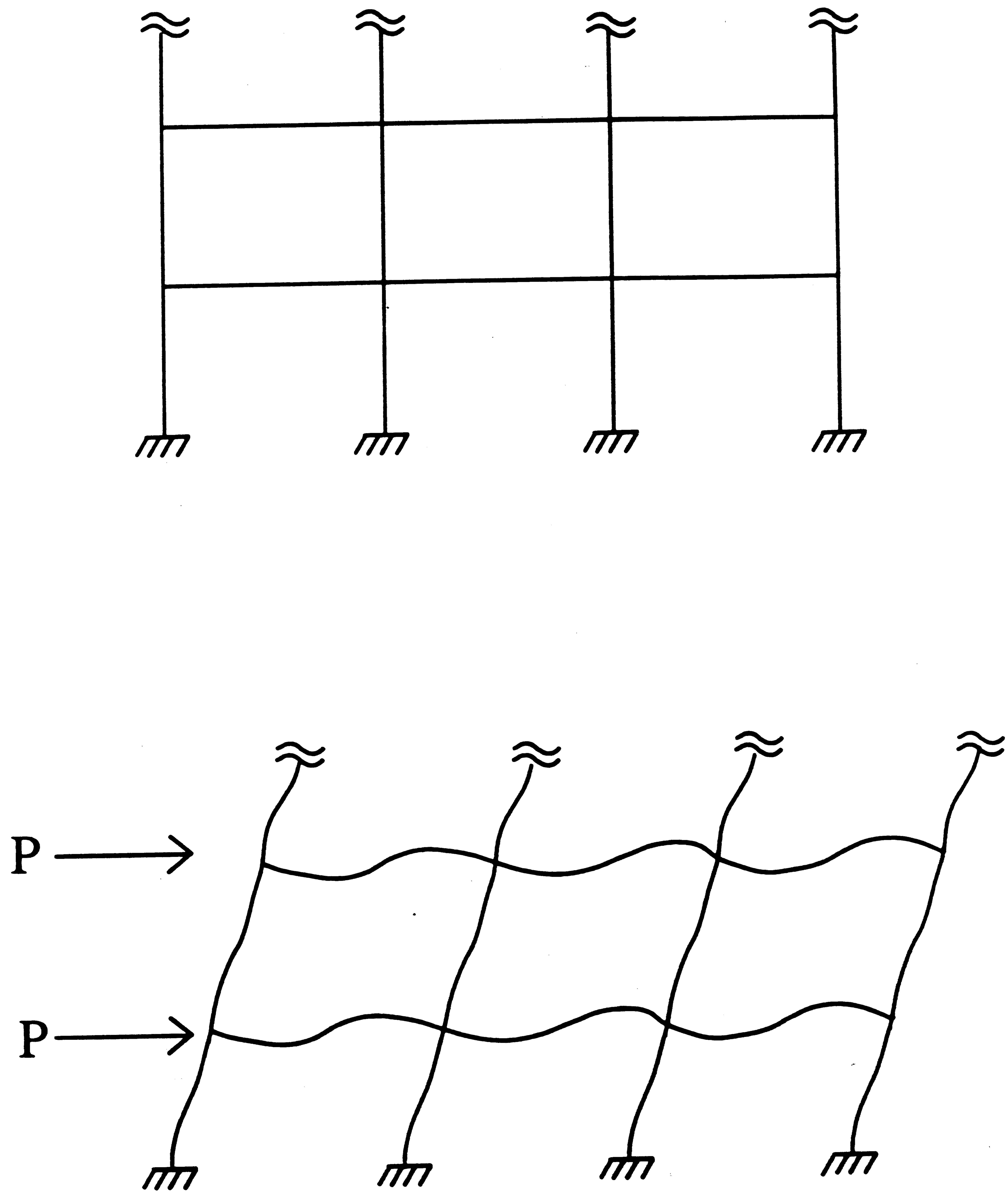


Figure 3-11: Lower Part of a Building With Fixed Base

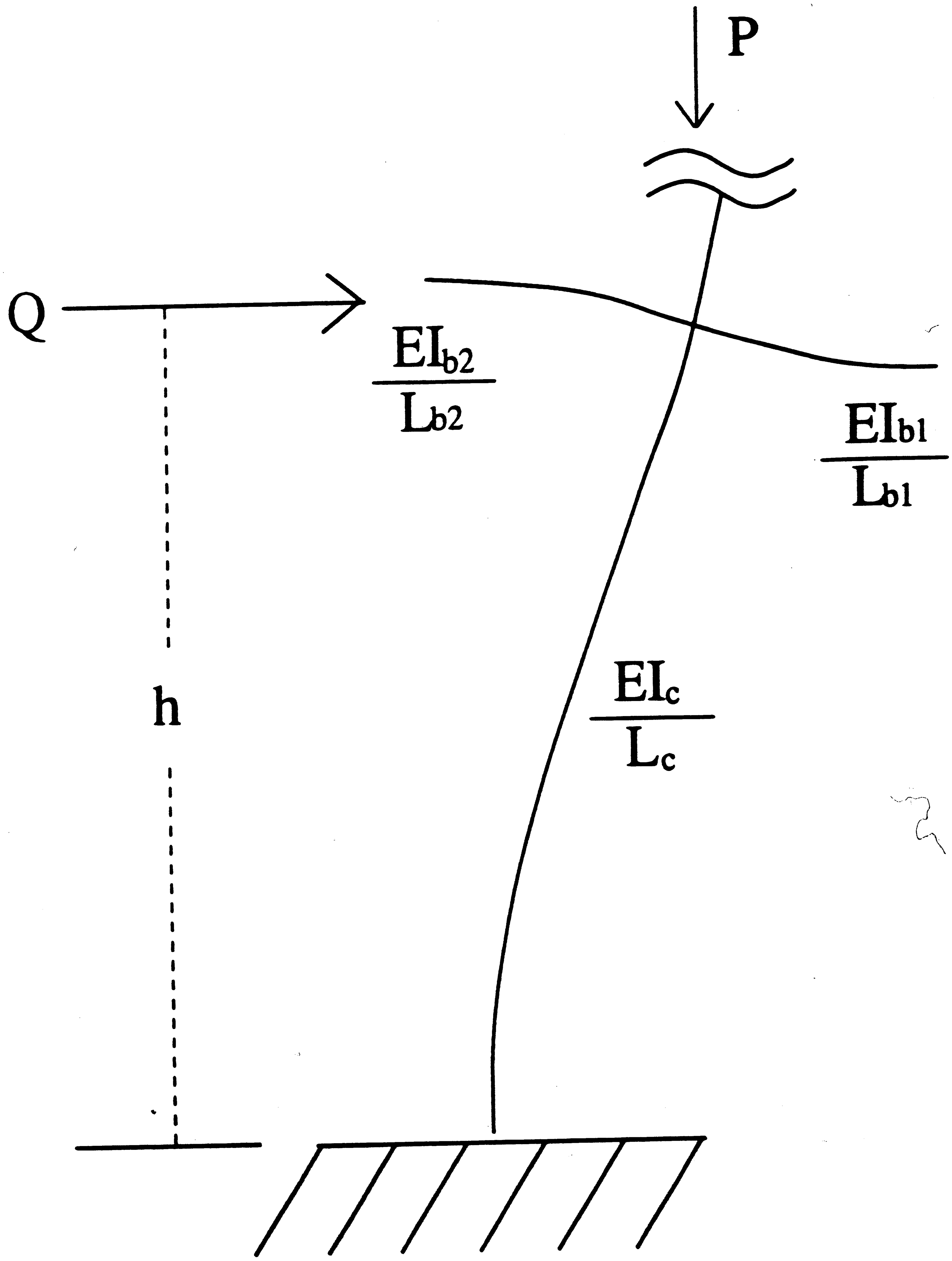


Figure 3-12: General Subassemblage in Bottom Story With Fixed Base

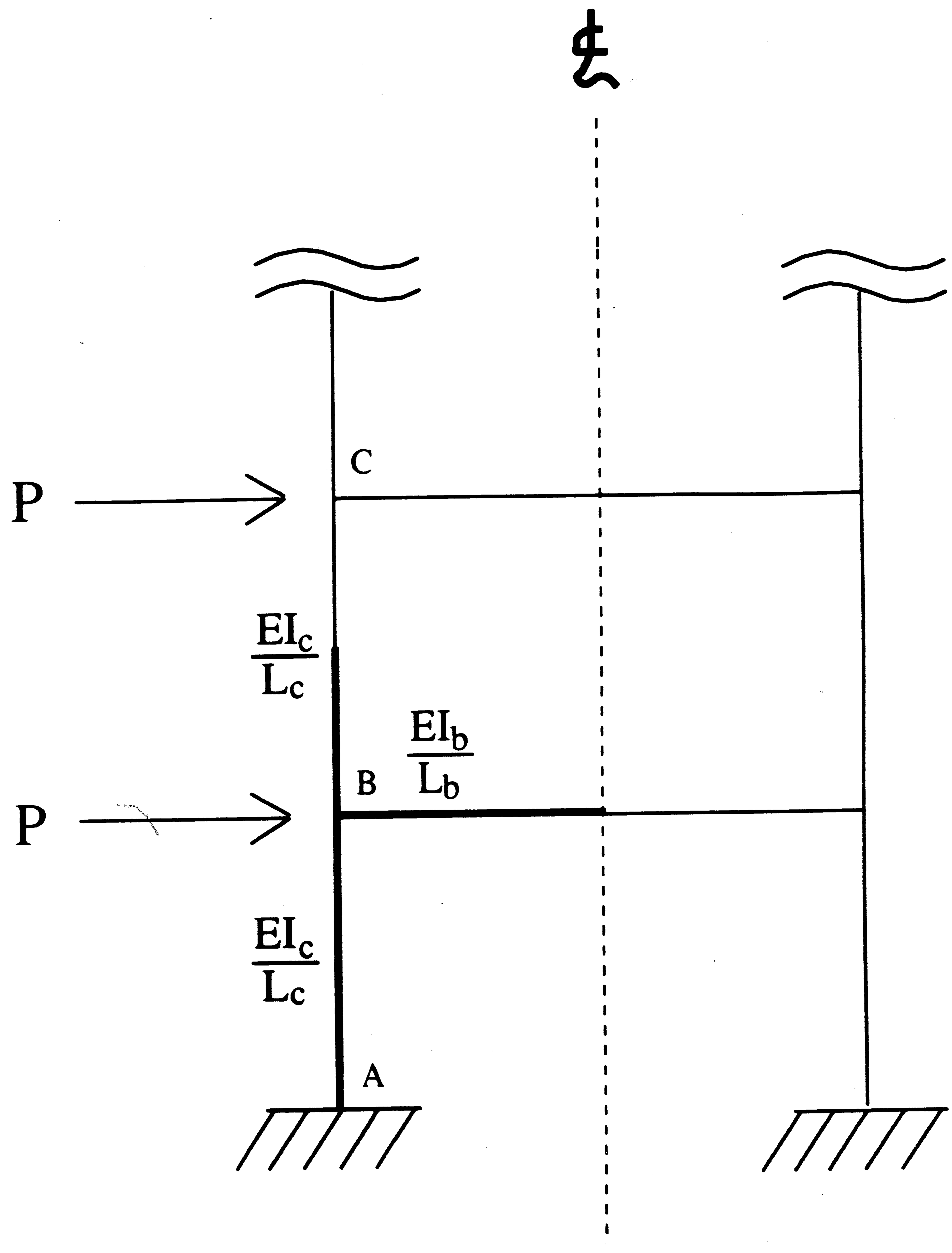
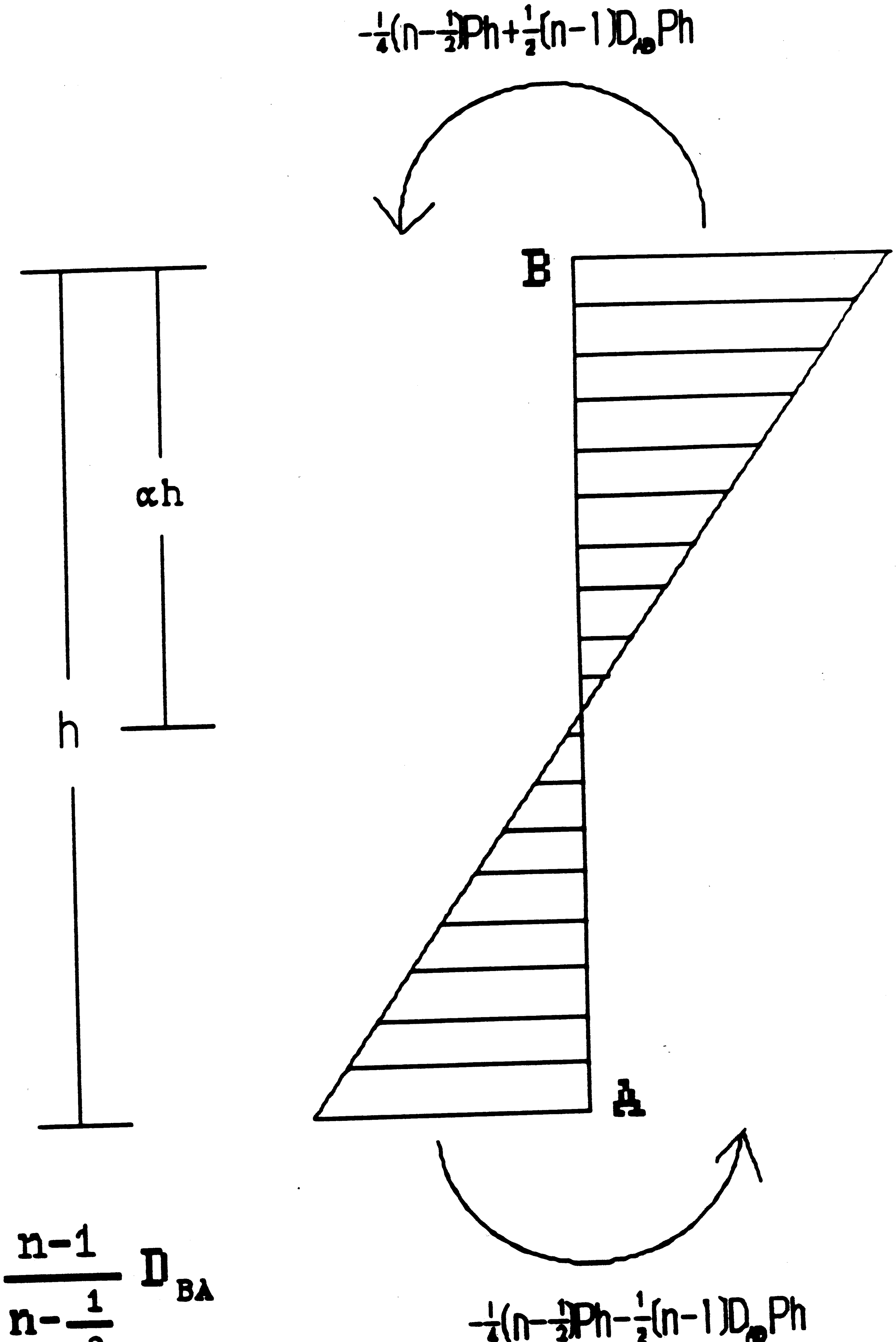


Figure 3-13: Bottom Story Subassemblage With Fixed Base Analysis Model



$$\alpha = \frac{1}{2} - \frac{n-1}{n-\frac{1}{2}} D_{BA}$$

Figure 3-14: Moment Diagram For Bottom Story Subassemblage with Fixed Base

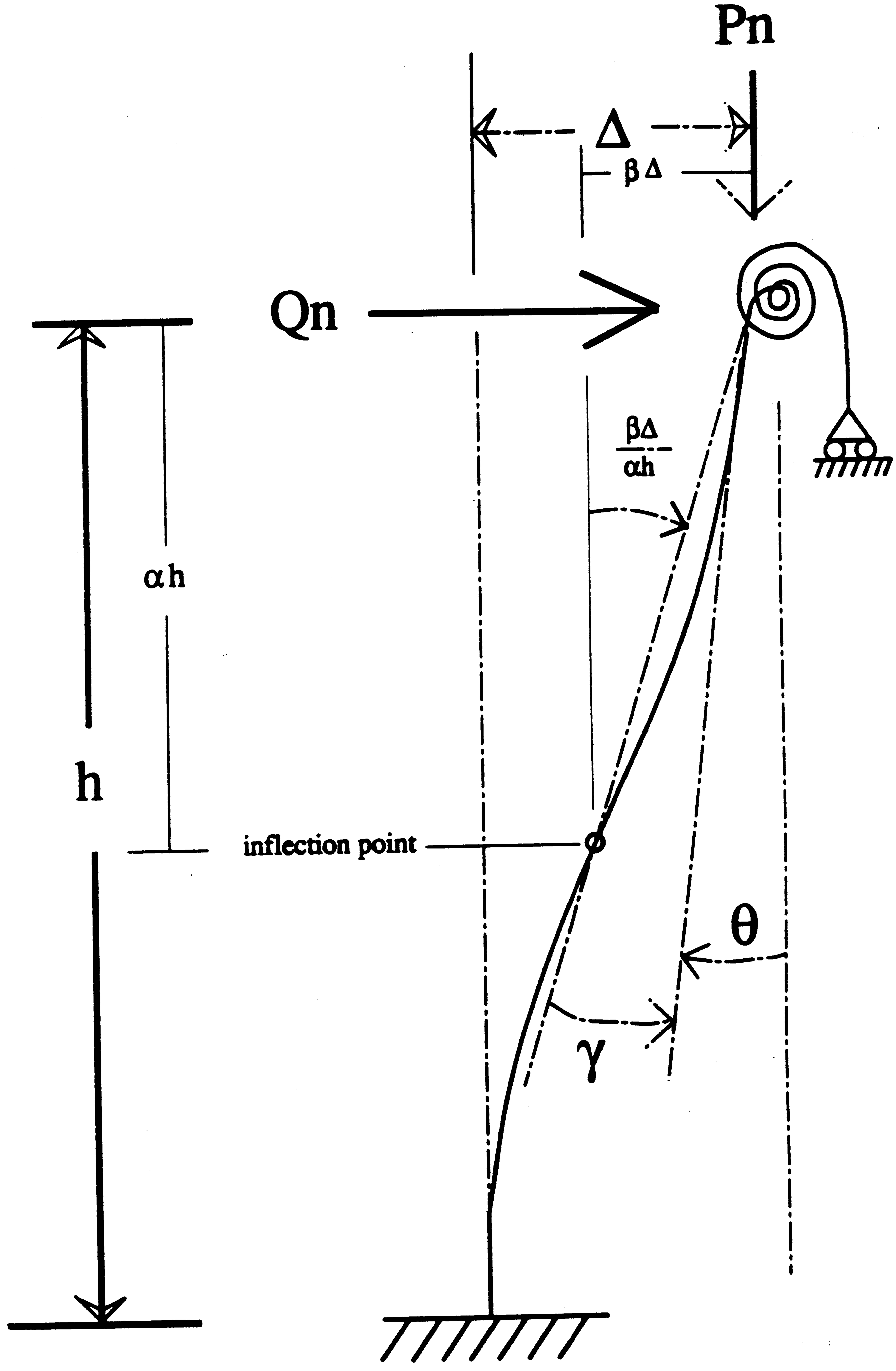


Figure 3-15: Subassemblage Model for Bottom Story with Fixed Base

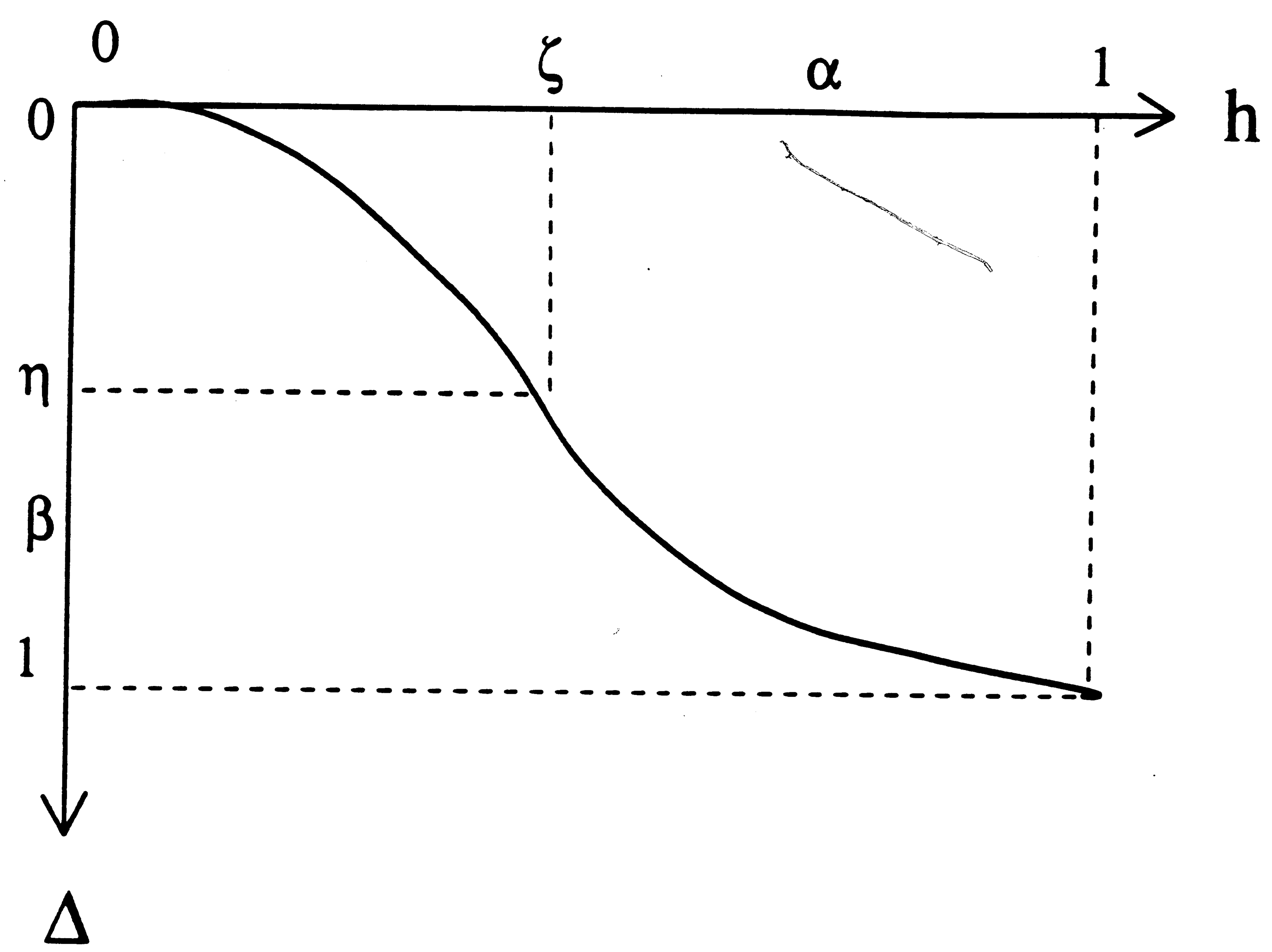


Figure 3-16: Approximation of A Subassemblage with Fixed Base

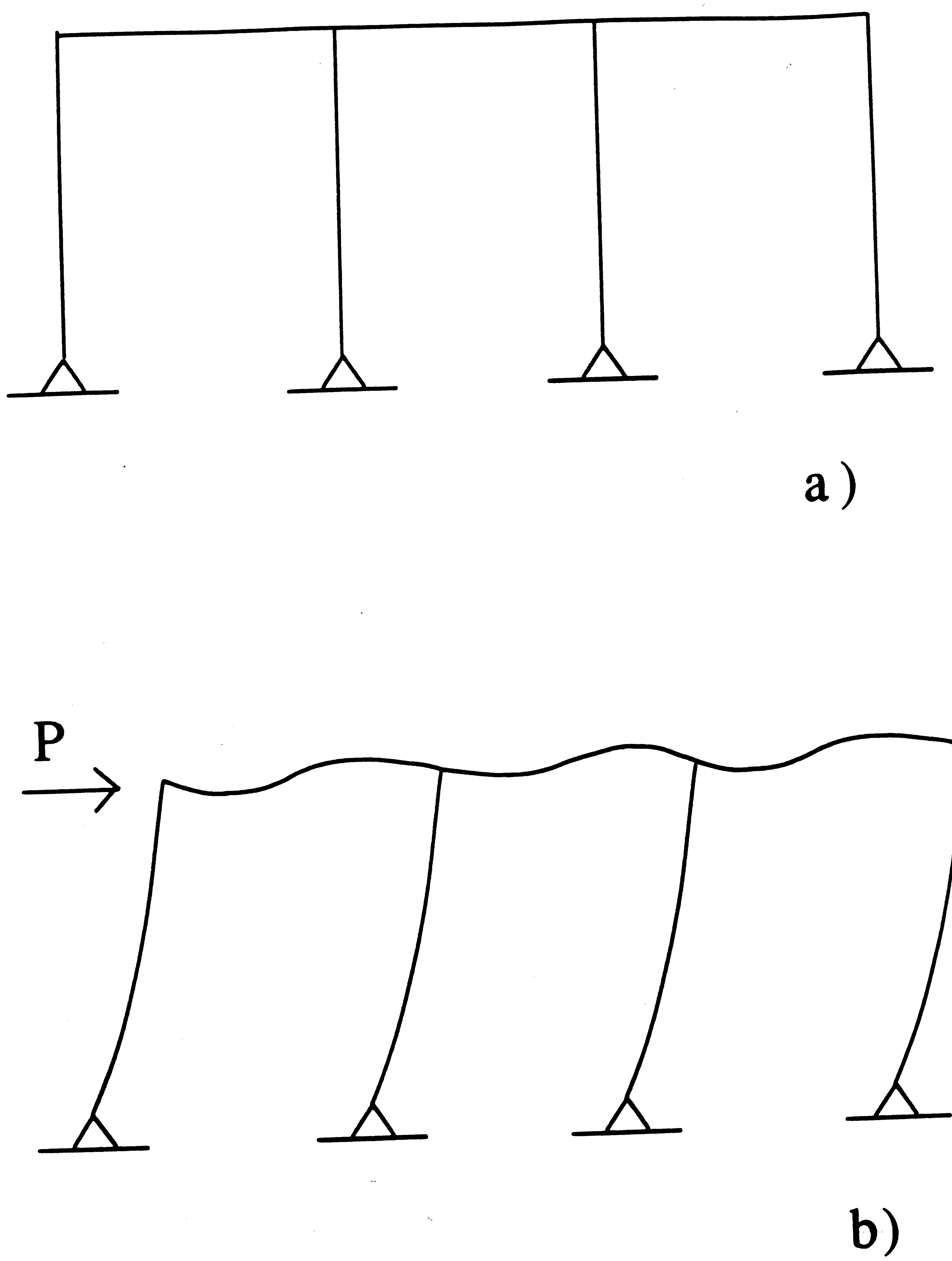
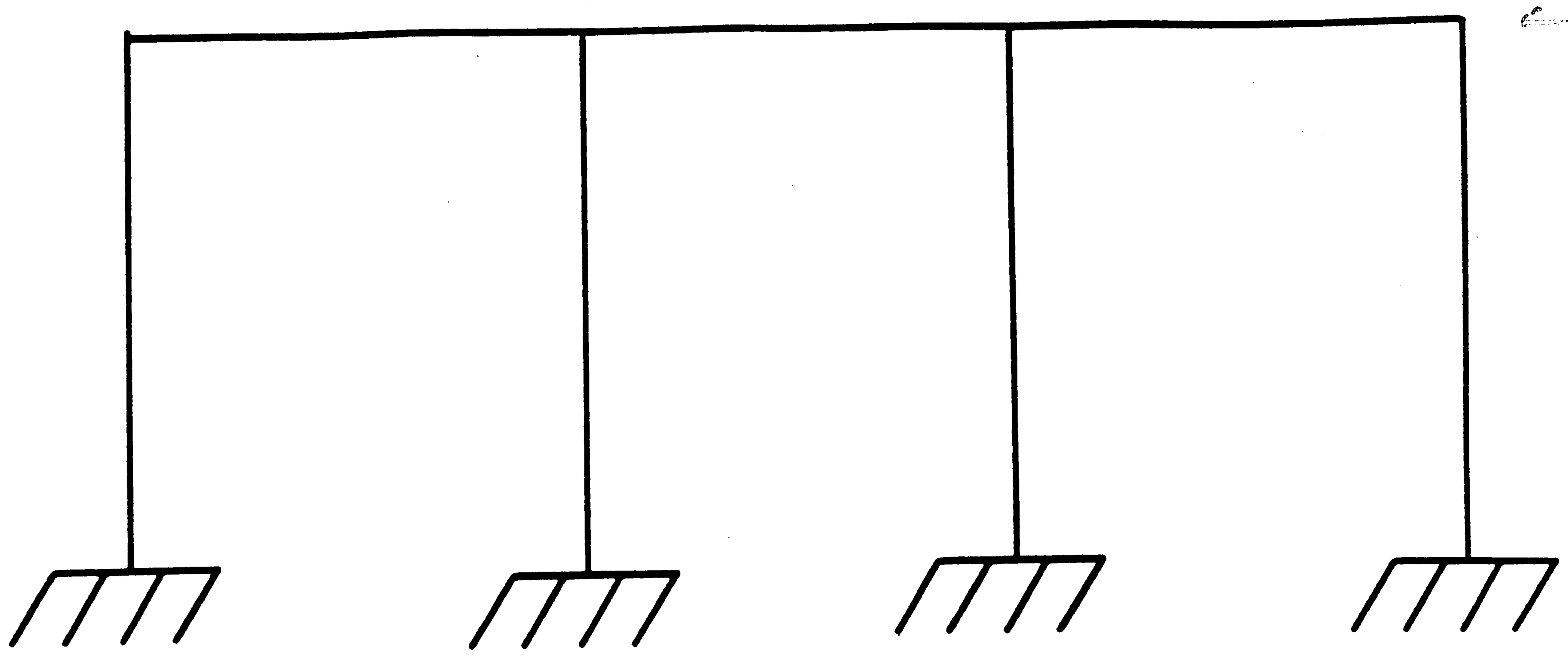
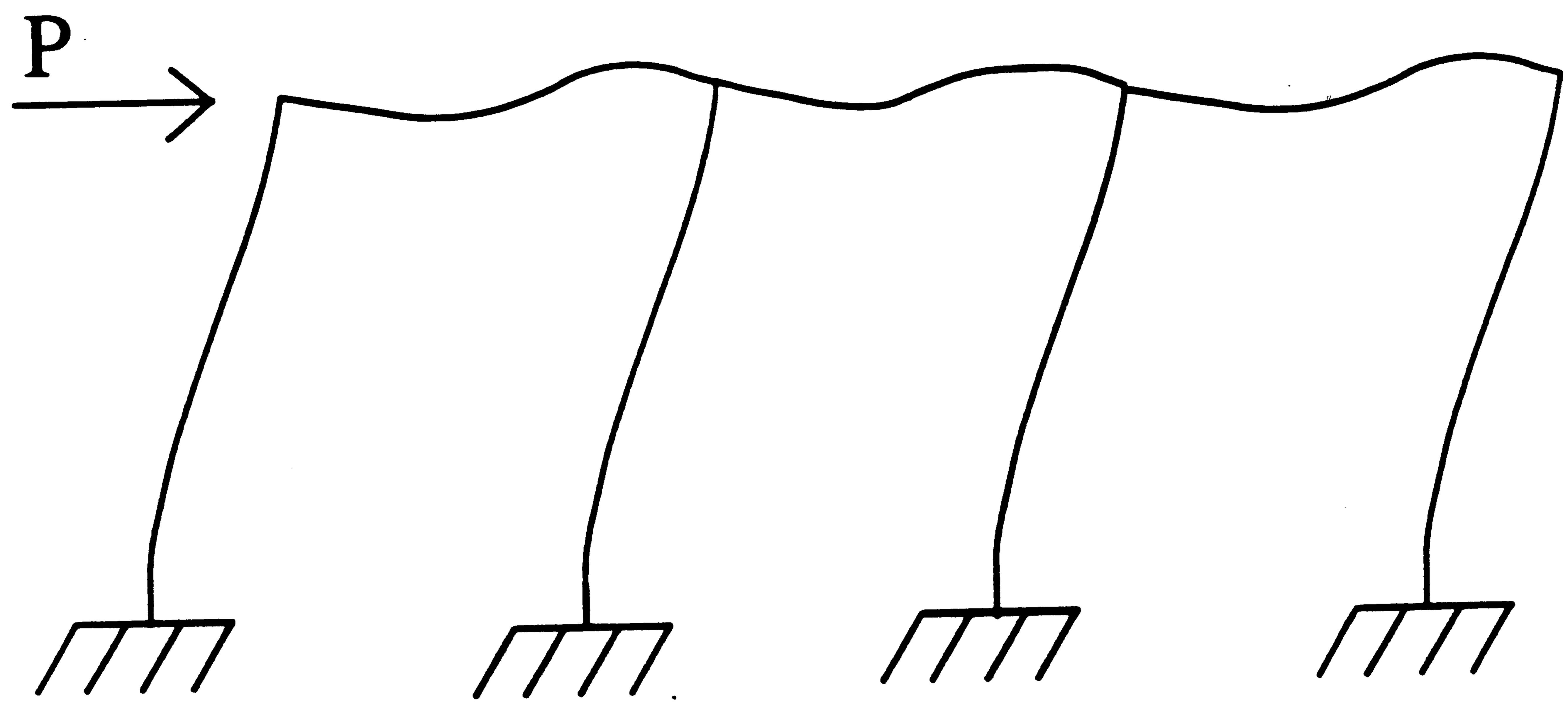


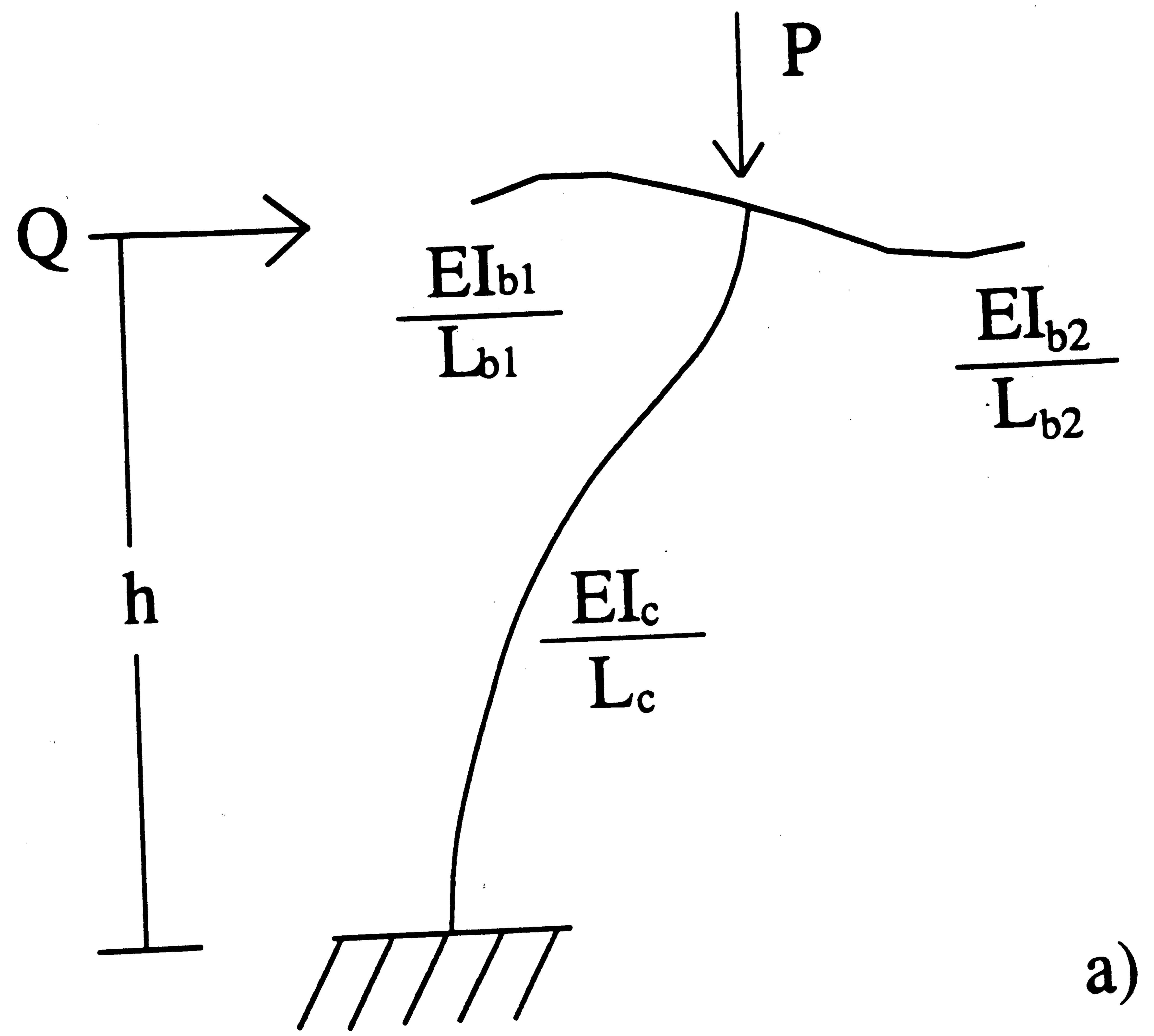
Figure 3-17: A Single-Story Building with Hinged Bases


a)



b)

Figure 3-18: A Single-Story Building with Fixed Bases



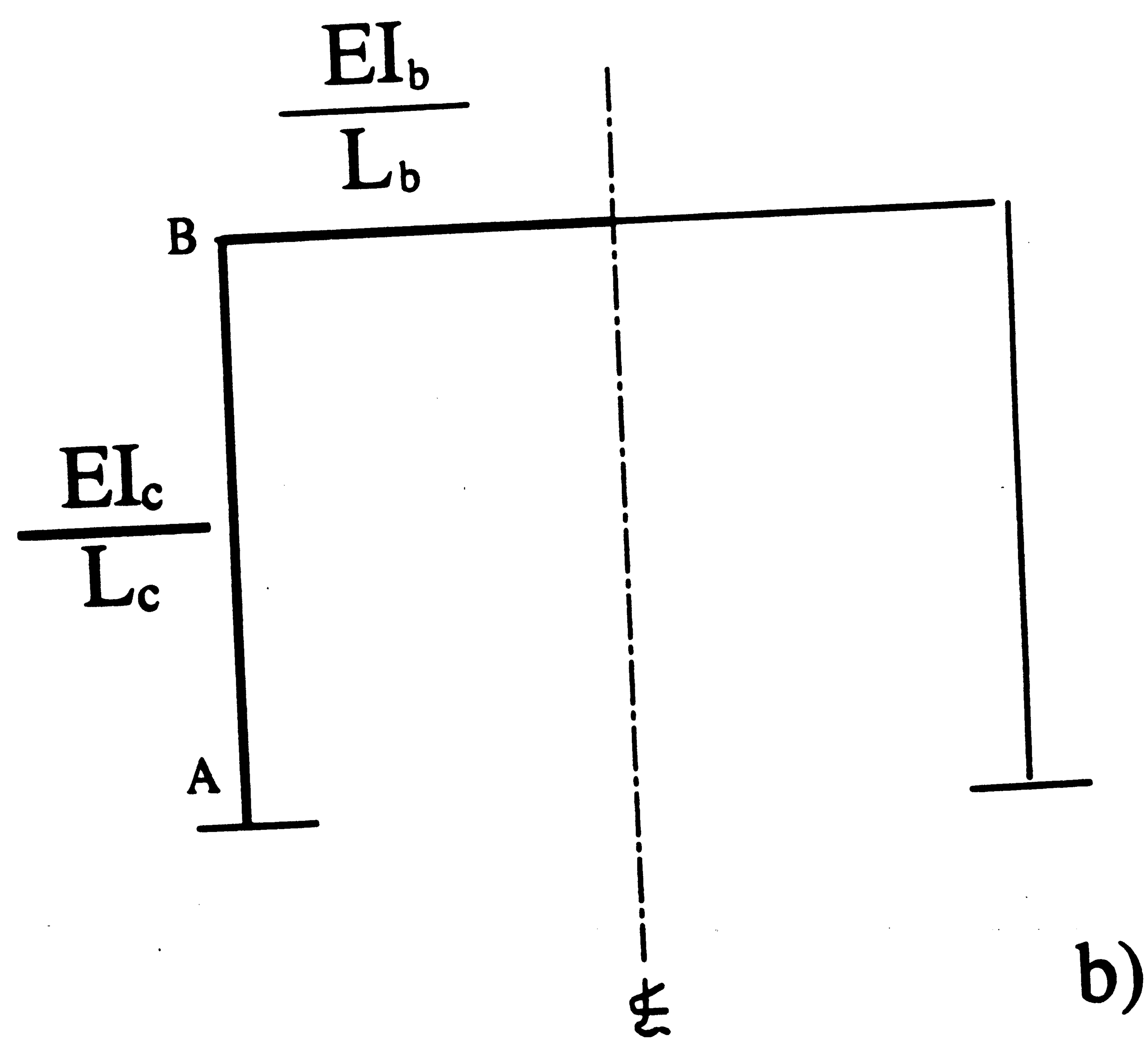


Figure 3-19: Single Story Subassemblage With Fixed Base Analysis Model

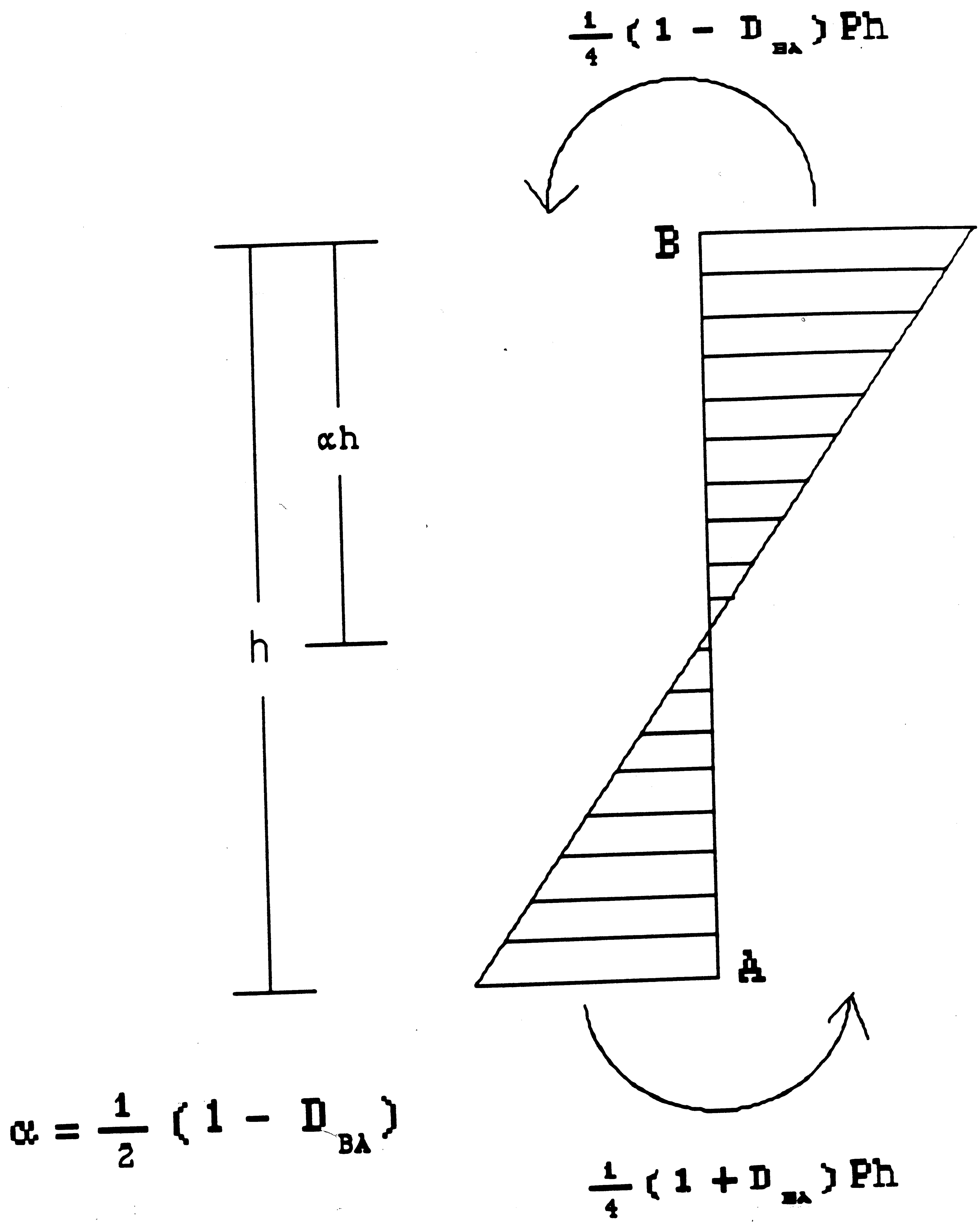


Figure 3-20: Single Story Subassemblage with Fixed Base Moment Diagram

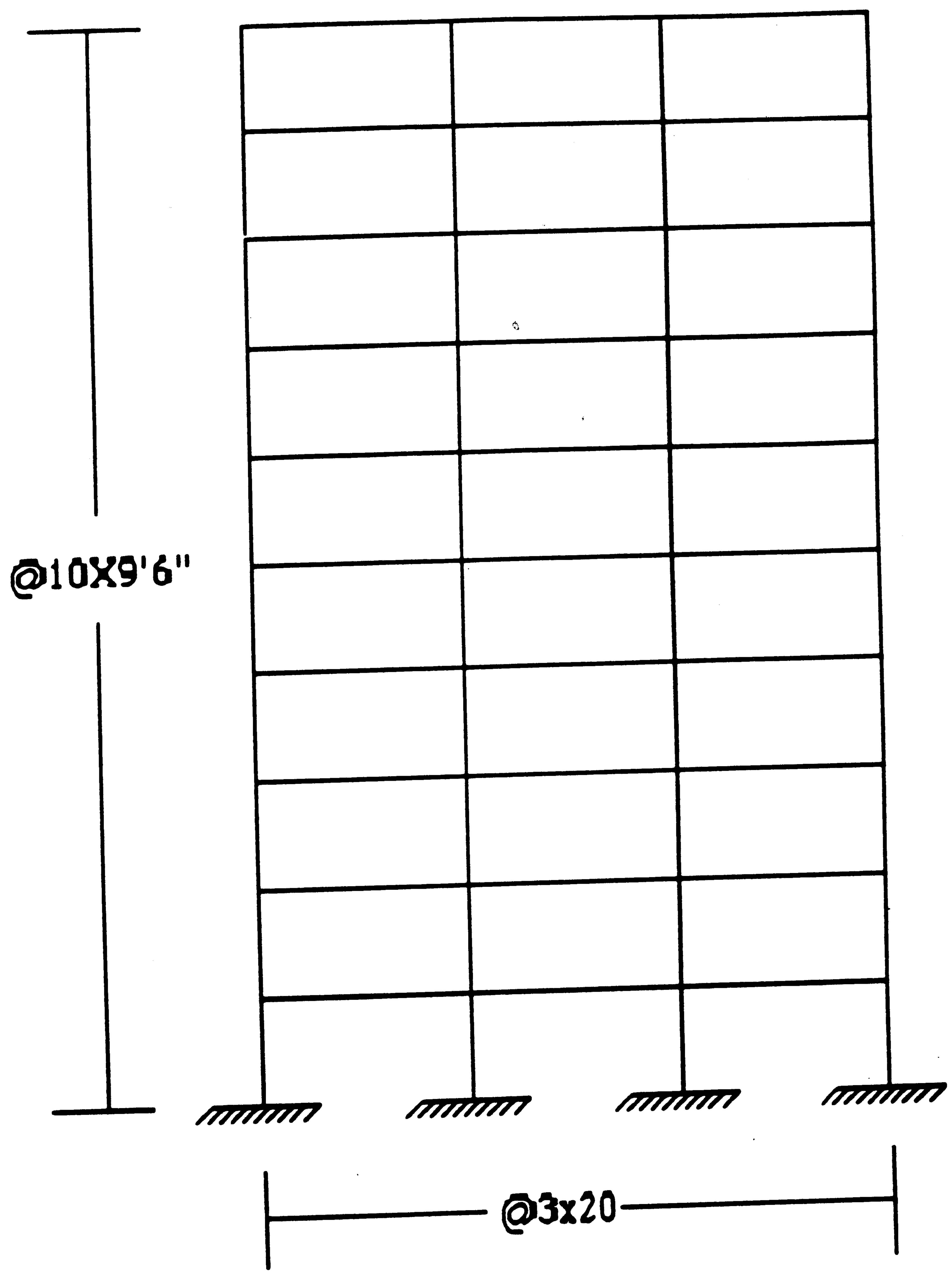


Figure 5-1: Ten Story Sample Building

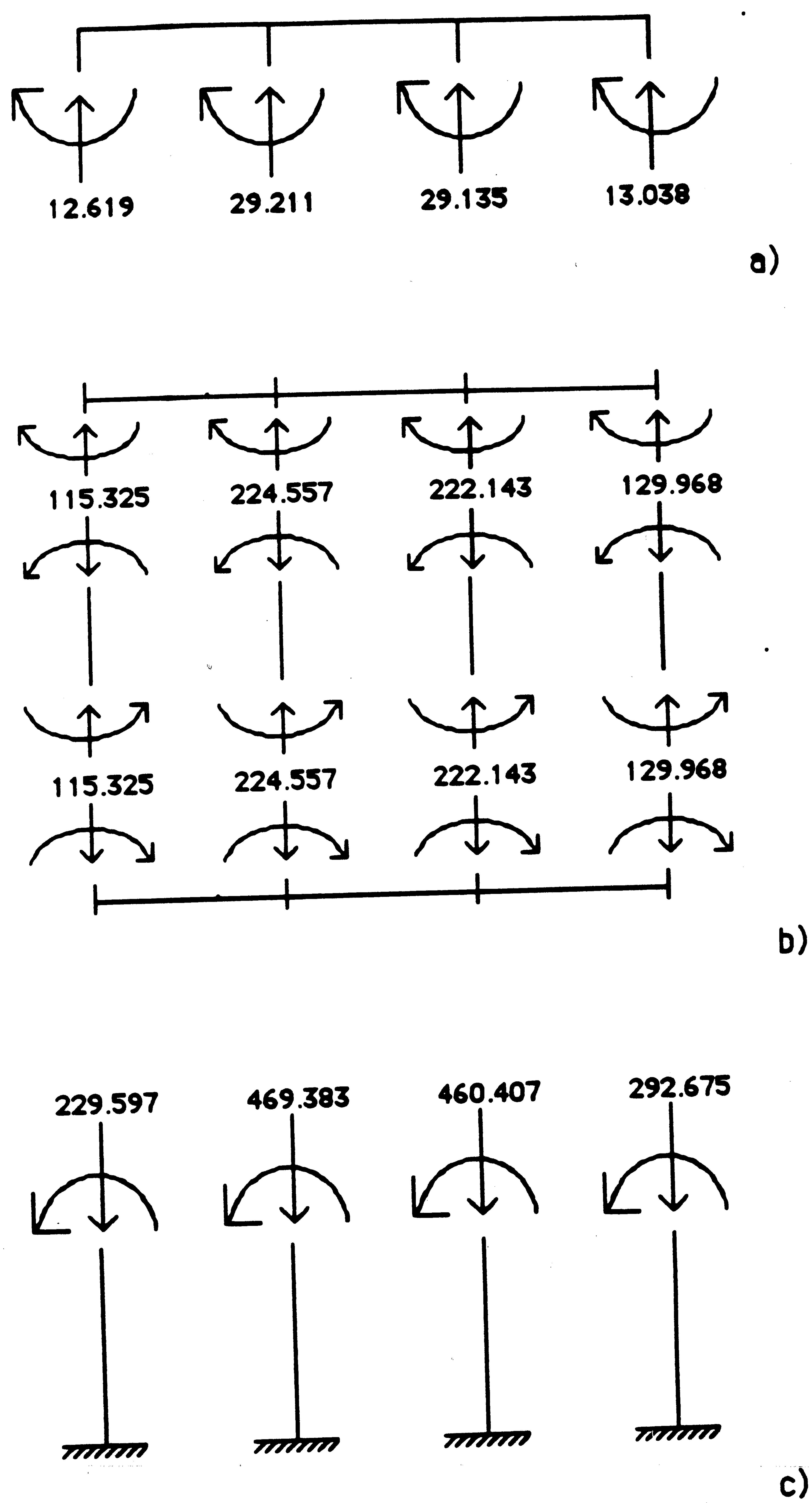


Figure 5-2: Free Body Diagrams for Top, Fifth and Bottom Floors

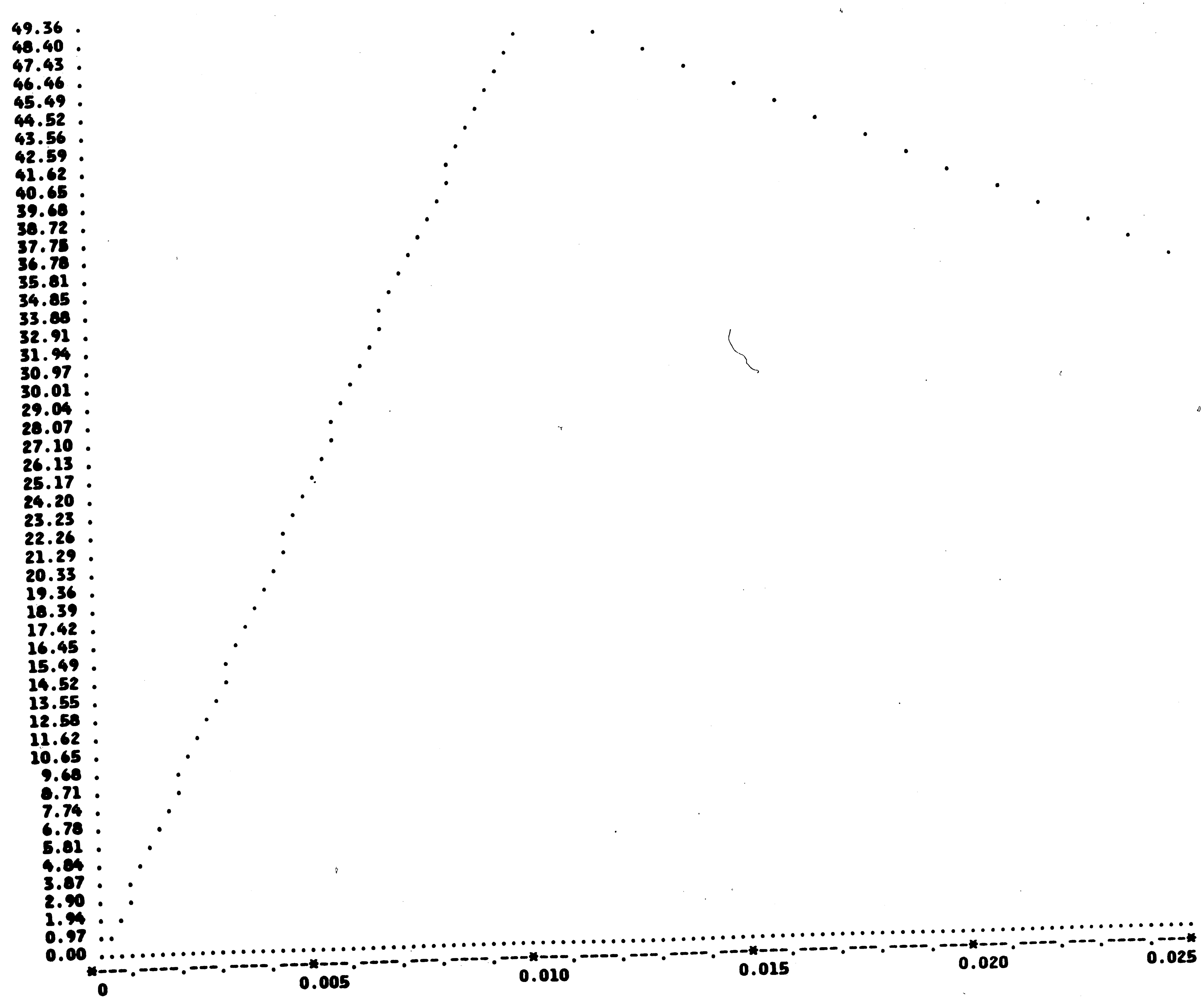


Figure 5-3: SMOA Output of Lateral Load-Deformation Figure for the Top Story

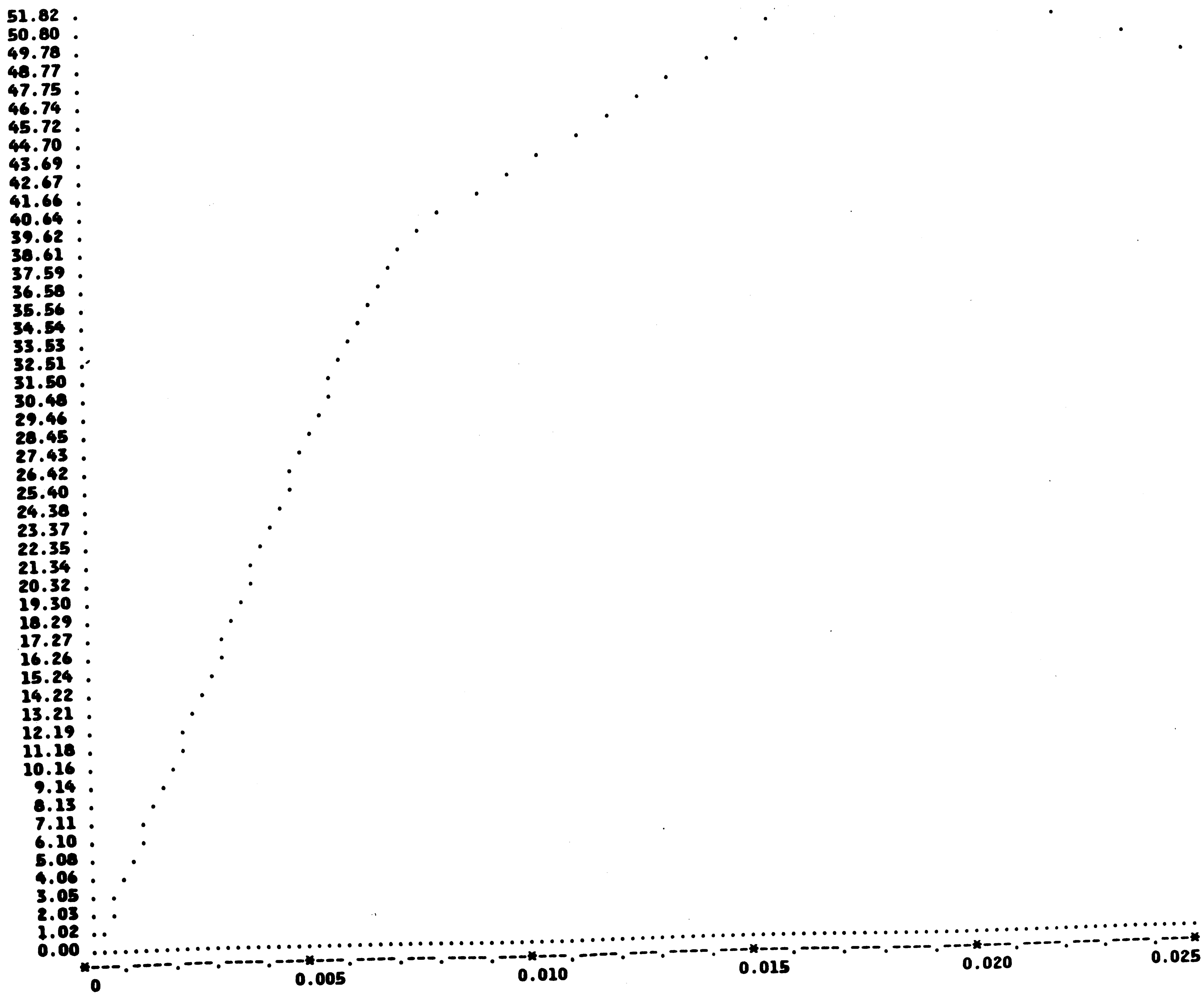


Figure 5-4: SMOA Output of Lateral Load-Deformation Figure for the Fifth Story

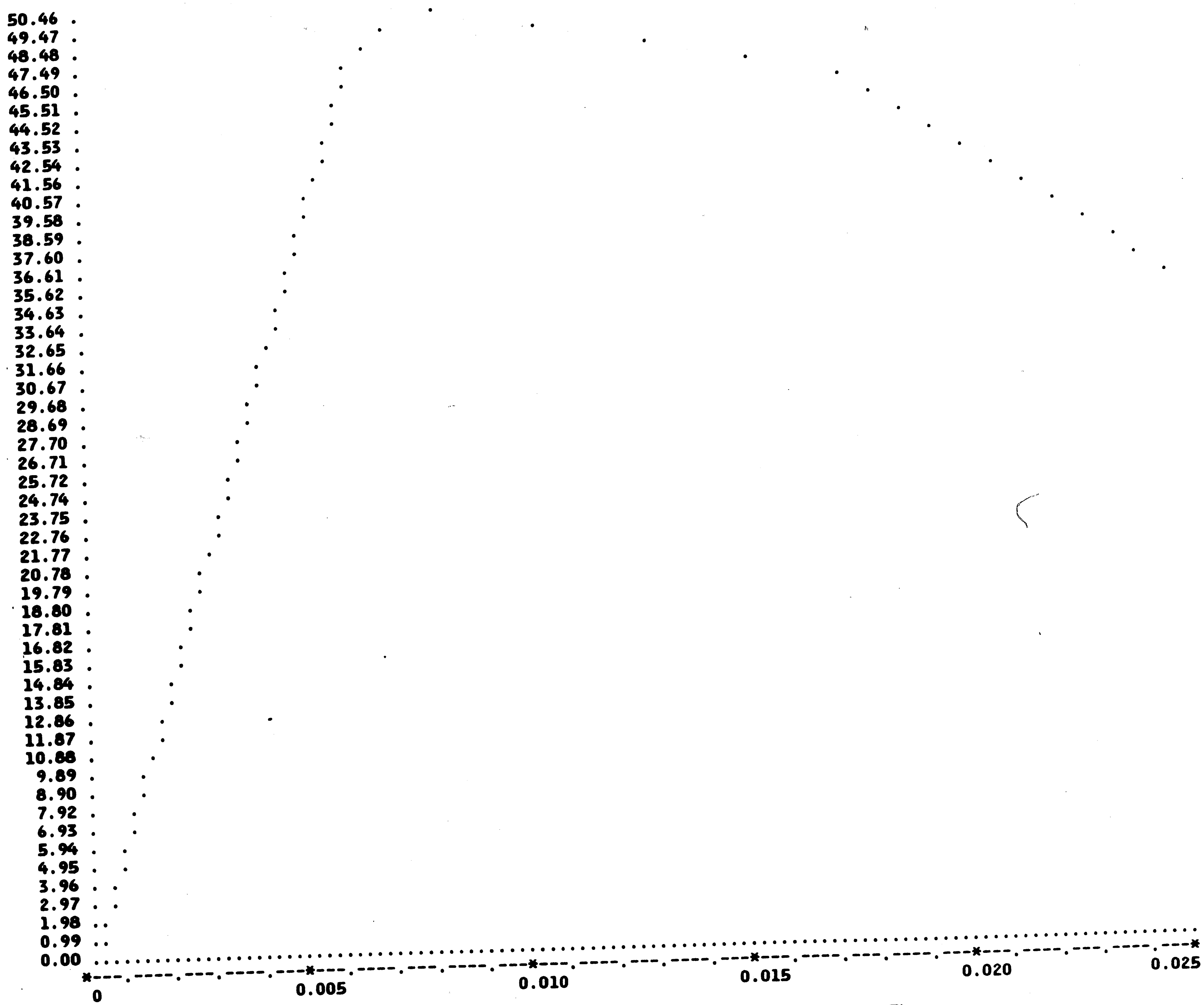


Figure 5-5: SMOA Output of Lateral Load-Deformation Figure for the Bottom Story

Nomenclature

THE MEANING OF THE PARAMETER NAMES IN THE PROGRAM " SMOA "

A	AREA
AADOH	RELATIVE STORY DEFLECTION OVER STORY HEIGHT FOR A PLASTIC HINGE ON THE SUBASSEMBLAGE CURVE
AAQ	HORIZONTAL LOAD AT PLASTIC HINGE POINT
AI	GIRDER Ixx
AIM1	INCREMENT OF MOMENT AT HINGE LOCATION ONE
AIM2	INCREMENT OF MOMENT AT HINGE LOCATION TWO
AIM3	INCREMENT OF MOMENT AT HINGE LOCATION THREE
AIM4	INCREMENT OF MOMENT AT HINGE LOCATION FORE
AITH	INCREMENT OF THETA
AKL	AVERAGE RESTRAINING STIFFNESS COEF. ON LEFT SIDE OF JOINT
AKR	AVERAGE RESTRAINING STIFFNESS COEFF. ON RIGHT SIDE OF JOINT
ALPHA	(I/L OF BEAM)/(I/H OF COL) LEFT COL AND RIGHT BEAM
ALPHP	(I/L OF BEAM)/(I/H OF COL) LEFT COL AND LEFT BEAM
AM	COLUMN END MOMENT
AMIL	INITIAL RESTRAINING MOMENT AT LEFT END OF GIRDER
AMIR	INITIAL RESTRAINING MOMENT AT RIGHT END OF GIRDER
AMP	GIRDER PLASTIC MOMENT
AMPC	MPC(reduced plastic moment) OF A COLUMN WHEN WIND FROM LEFT
AMPCR	MPC OF A COLUMN UNDER LOAD WHEN WIND IS FROM RIGHT
AMP1	LIMITING MOMENT AT RIGHT END OF GIRDER(M1)
AMRL	RESTRAINING MOMENT ON LEFT SIDE OF JOINT
AMRLY	RESTRAINING MOMENT ON LEFT SIDE OF JOINT AT THE FORMATION OF A PLASTIC HINGE AT Y
AMPM	MINIMUN PLASTIC MOMENT TO RESIST 1.3 TIMES THE WORKING GRAVITY LOAD (FL 273.20, eq.14.7)
AMRR1	RESTRAINING MOMENT ON RIGHT SIDE OF JOINT AT THE FORMATION OF A PLASTIC HINGE AT 1
AMRR	RESTRAINING MOMENT ON RIGHT SIDE OF JOINT
AM1	MOMENT AT RIGTH END OF RIGHT BEAM

AM2	MOMENT AT LEFT END OF RIGHT BEAM
AM3	MOMENT AT RIGHT END OF LEFT BEAM
AM4	MOMENT AT LEFT END OF LEFT BEAM
B	FLANGE WIDTH
BETA	(I/L OF BEAM)/(I/H OF COL) RIGHT COL AND RIGHT BEAM
BIOL	THE TOTAL I/L OF THE MODEL BEAM
BKL	RESTRAINING STIFFNESS COEFF. ON LEFT SIDE OF JOINT
BKR	RESTRAINING STIFFNESS COEFF. ON RIGHT SIDE OF JOINT
BL	BEAM LENGTH CENTER-TO-CENTER OF SUPPORTS
BLG	CLEAR SPAN LENGTH OF BEAM
CI	Ixx OF COLUMN
CIOL	I/L OF THE COLUMN IN THE SUBASSEMBLAGE
CJ	TORSION CONSTANT FOR SECTION = J
CKL	RESTRAINING STIFFNESS COEFF. ON LEFT SIDE OF JOINT
CKR	RESTRAINING STIFFNESS COEFF. ON RIGHT SIDE OF JOINT
COEF	ASTM A6 STRUCTURAL SHAPE GROUP
D	DEPTH
DC	DEPTH OF COLUMN
DELTATH	THE POSSIBLE ROTATION OF A JOINT TO LET MOMENT OF THE JOINT BECOME A HINGE
DOH	RELATIVE HORIZONTAL STORY DISPLACEMENT DIVIDED BY THE STORY HEIGHT
ETA	(I/L OF BEAM)/(I/H OF COL) RIGHT COL AND RIGHT BEAM
FN	A DUMMY VARIABLE TO REDUCE M FROM 1 TO 0.001
FY	YIELD STRESS OF STEEL
GAM	COLUMN ROTATION BASED ON STRAIGHT LINE MOMENT-ROTATION CURVE
GAP	COLUMN ROTATION WHEN COLUMN MOMENT EQUALS THE COLUMN PLASTIC MOMENT
H	STORY HEIGHT
HC	HINGE COUNTERS FOR COLUMN. 0=NO HINGE 1=HINGE
HX	HC DIST. X FROM RIGHT END OF RIGHT GIRDER
HY	HC DIST. Y FROM RIGHT END OF LEFT GIRDER
H1	HC RIGHT END OF RIGHT GIRDER
H2	HC LEFT END OF RIGHT GIRDER
H3	HC RIGHT OF LEFT GIRDER

H4	HC LEFT END OF LEFT GIRDER
I	NUMBERS OF ROW IN ARRAY FOR EACH SUBASSEMBLAGE
IHINGE	INDICATE THE NEXT POINT TO BECOME A HINGE IN A MECHANISM
IND	COUNTER . 1 = WIND FROM LEFT 2= WIND FROM RIGHT
J	NUMBER OF JOINTS, COLUMN BELOW AND GIRDER TO RIGHT OF JOINT
J1	SUBSCRIPT FOR A GIVEN COLUMN
J4	COUNTER FOR GIRDERS
J5	COUNTER FOR COLUMNS
K	NUMBER OF FLOOR LEVEL TO BE STUDIED
KF	FRACTIONAL PART OF FILLET DEPTH
KI	INTEGER PART OF FILLET DEPTH (IN)
K1	COUNTER FOR WHICH LEVEL IS BEING CALCULATED
L	NUMBER OF BAYS
LEV	LEVEL NUMBER AS IDENTIFIED FOR A GIVEN STRUCTURE
M	NUMBER OF ROWS IN ARRAY FOR EACH SUBASSEMBLAGE
N	HINGE NUMBER IN THE ORDER OF FORMATION
NFF	COUNTER WHICH LABELS THE DATA AS GIRDER LOADS
P	FACTORED LOAD ON A COLUMN
PL	FACTORED LOAD ON A COLUMN WITH WIND FROM LEFT
PR	FACTORED LOAD ON A COLUMN WITH WIND FROM RIGHT
PY	COLUMN YIELD LOAD
Q	HORIZONTAL LOAD
RCM	RESTRAINED COLUMN END MOMENT
RM	RESTRAINING MOMENT AT JOINT
RMA	ABSOLUTE VALUE OF RESTRAINING MOMENT
RT	RADIUS OF GYRATION OF FLANGE/WEB TEE (IN)
RX	COLUMN X-X RADIUS OF GYRATION
RYY	RADIUS OF GYRATION ABOUT Y AXIS
SXX	SECTION MODULUS ABOUT X AXIS
SYY	SECTION MODULUS ABOUT Y AXIS
T	FLANGE THICKNESS
TH	INCREMENT OF THETA (RADIANS)
THE	SUM OF THE JOINT ROTATION

THEY	SUM OF JOINT ROTATION AT THE FORMATION OF A PLASTIC HINGE AT Y
THE1	TOTAL ROTATION AT FORMATION OF A PLASTIC AT RIGHT END OF RIGHT BEAM. (HINGE 1). IT STOPS INCREASING WHEN HINGE FORMS
THE(J)	SUM OF JOINT ROTATION FOR JOINT J
W	UNIFORMLY DISTRIBUTED BEAM LOAD
WQ	WEB THICKNESS
XXI	MOMENT OF INERTIA ABOUT X AXIS
X1	THE DISTANCE OF HINGE X TO HINGE 1 IN EACH SUB-ASSEMBLAGE
YYI	MOMENT OF INERTIA ABOUT Y AXIS
Y1	THE DISTANCE OF HINGE Y TO HINGE 3 IN EACH SUB-ASSEMBLAGE
Z	PLASTIC SECTION MODULUS

Appendix A
A Sample Input File for SMOA

```
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36.00   36.00                                                                       (column loads when wind from left )
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W 24084002470241009020077004702310106023700196000979094402090195003700224003 2602
                                                         (one way analysis)  (floor No.)  (interior floor)
   1    2    2                                           (height)
12.00                                                    (beam spans)
20.00   12.00   20.00                                    (Fyc, Fyb)
36.00   36.00                                                                       (column loads when wind from left)
  1757.   1836.   1836.   1757.    0.    0.    0.    0.    0.                         (beam loads)
   6.86    6.86    6.86    0.00   0.00   0.00   0.00   0.00      2 24                 \
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Appendix B
Input File for Chapter 5

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36.00   36.00
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20.00   20.00   20.00
36.00   36.00
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Vita

Shih Hsun Yuan, born on July 19,1963 in Taipei, Taiwan, R.O.C. and raised in Taipei, is the only son of Mr. Chao-Fan Yuan and Mrs. Pan-Shi Chen.

After graduating from National Chung-Hsing University where he was awarded the his B.S. degree in Civil Engineering, he was in the engineering corps of the army as a second lieutenant for two years.

After completing military service, he came to Lehigh University in August 1988 as a master's candidate.